SRR-89-03

Load Testing and Evaluation of the Glen Allan Bridge

Ontario

Ministry of Transportation

Research and Development Branch

Technical Report Documentation Page

Load Testing and Evaluation of the Glen Allan Bridge

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Number:	SRR-89-03
Date of Publication:	May 1989
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Abstract:	This report describes the load testing and evaluation of the Glen Allan Bridge. The bridge was proof tested in 1986 as part of MTO's on-going bridge testing program, in order to derive a realistic load carrying capacity. The bridge is comprised of a steel through truss with a concrete deck on steel stringers and floor beams. It was found to be considerably substandard by normal evaluation procedures. Load testing confirmed the validity of utilizing a frame analysis to model the trusses and demonstrated continuity in the deck system which increased the stringer capacity.
Comments:	
Key words:	bridge testing, steel truss, deteriorated steel truss
Distribution:	MTO Head Office and Regions, key Ministry officials
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SRR-89-03

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Published by
The Research and Development Branch
Ontario Ministry of Transportation
Hon. Ed Fulton, Minister
D.G. Hobbs, Deputy Minister

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May 1989

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Test Summary

Bridge:	Glen Allan Bridge, Wellington County Rd. 45, Site No. 35-240
Type of Structure:	Steel Box Truss with concrete deck slab on steel stringers and floor beams.
Posted Load:	9 tonnes
Span/Roadway Width:	Total span 49 m/5 m
Angle of Skew:	43°
Date of Test:	June 5, 1986
Type of Test:	Proof
Maximum Static Load Applied:	Single test vehicle (Kenworth) applied with maximum static load of 77 tonnes.
Responses Measured:	Strains at critical sections of truss members, stringers and floor beams. Midspan deflection of south truss.

1/ Introduction

The Glen Allan Bridge was evaluated in 1985 in accordance with the Ontario Highway Bridge Design Code (OHBDC) [1]. This evaluation resulted in recommendations for considerable rehabilitation in order to prevent severe posting restrictions. Due to the varied and sometimes extensive levels of deterioration of both the deck and truss members, the traditional evaluation process required conservative approximations for member capacities.

Because there are potential savings associated with a reduction in the rehabilitation requirements the County of Wellington requested that the structure be load tested by the Ministry of Transportation (MTO). The bridge was included in MTO's 1986 program and was tested on June 5, 1986. The preliminary report recommended a triple posting of 15, 22 and 28 tonnes under the current structure conditions considering that repairs to severely deteriorated members would be undertaken within the next year.

2/ Site Description

The Glen Allan Bridge shown in Figure 1 is a single-lane structure located on Wellington County Road 45 at Glen Allan in Peel Township. This single-span structure is identified as MTO Site No. 35-240 and has a span of 48.46 m over the Conestogo River. The Pratt steel trusses support a reinforced concrete deck on steel stringers and steel floor beams.

The dimensional layout of the deck system is provided in Figure 2, with a typical cross-section given in Figure 3. Figure 4 provides overall dimensions of the steel trusses along with the section details of the members. All dimensions were measured in the field as the drawings provided from the MTO file were suspect since a preliminary dimension check displayed some discrepancies.

Field inspection of the bridge showed varied levels of deterioration in both the deck and trusses with some members having up to 50% section loss. The steel floor beams are in fair condition with deterioration near the ends of up to 20%. The interior steel stringers are in fair condition but the exterior stringers are poor with heavy deterioration of the webs and bottom flanges of between 25 to 30% section loss. The worst stringers are those in the end bays supported at the abutments.

The steel truss members have varied deterioration levels with an overall section loss of about 15% and local member deterioration as given in Figures 5 and 6. These section losses were approximated through visual examination and the most critical value was used in subsequent member resistance calculations for a particular member type.

The concrete deck is in poor condition with numerous areas of active spalling and exposed deteriorating reinforcement. There is evidence of some cracking and leeching on the underside of the deck although there is no sign of structural failure. The concrete deck slab is not touching the top flanges of the exterior stringers in nearly all cases as depicted in Figure 3; sometimes with a gap of over 25 mm.

3/ Preliminary Analysis and Evaluation

The preliminary evaluation of the structure was performed in accordance with the OHBDC [1]. The simplified method of analysis was applied to the evaluation of the deck system and a frame analysis [2] was used to determine member forces in the steel trusses.

3.1/ Deck System

The evaluation of the stringers and floor beams assumed non-composite action between the concrete deck and the steel supporting members. A 10% deterioration of the stringers and floor beams was assumed.

The critical scale-down factors derived in accordance with the OHBDC were F = 0.70 and F = 0.74 for the stringers and floor beams, respectively.

3.2/ Steel Trusses

The steel trusses were analysed as two-dimensional frames, as depicted in Figure 7, using STRUDL [2] . This analysis was used to determine dead load axial forces and to generate influence coefficients for the axial forces in all members under a unit load applied at each node along the bottom chord.

These member influence lines were used in conjunction with an in-house influence line program to determine critical member axial forces under the factored OHBDC design loads. The factored axial member resistances were determined for each member according to the OHBDC assuming F_y = 230 MPa and an effective length factor of K = 1.0 for compression. The assumed member deteriorations, as given in Figures 5 and 6, were incorporated into the calculated resistances according to the following considerations:

i) Compression Members, lesser of;
   - overall buckling capacity reduced by 5%
   - observed reduction (Figures 5 and 6) applied to AF_y

ii) Tension Members, lesser of;
   - net section capacity reduced by 15%
   - observed reduction (Figures 5 and 6) applied to gross section capacity

Table 1 gives the results of this evaluation in terms of capacity reduction factors for the three evaluation levels.

The zero compressive resistance for member 31 was assumed because the calculated slenderness of the member exceeded the limits of the formulæ in the OHBDC [1]. Members 38 and 39, are very slender also and therefore have very low compressive capacities.

In order to evaluate the effects of the potential buckling of members 38 and 39 the frame analysis was re-run ignoring each of these members for certain loadings. New influence lines were derived using the results of this new analysis only through the range of vehicle positions which would actually cause members 38 and 39 to be in compression. The results of a re-evaluation using this analysis are provided in Table 2. The only members that are affected by the change in analysis are members 32 and 33. None of the other member results are changed and member 31 still governs the truss capacity for all three levels.

3.3/ Summary

Table 3 provides a summary of the critical scale-down factors for the stringers, floor beams and trusses for all three evaluation levels. This summary ignores members 38 and 39 in compression. It also assumes no axial compressive resistance for member 31 which governs the truss members.

Using these values the required triple posting would be 15, 24 and 28 tonnes governed by the stringers.

Additional evaluation was to be performed following the load testing.

4/ Instrumentation

The bridge was instrumented to monitor steel strains in both the deck system and steel trusses. All of the 60 channels of MTO's Motorola 6800 computer-based data acquisition system were used.

4.1/ Deck System

Critical flexural strains were monitored on the first two bays of the deck system from the east end of the bridge, as shown in Figure 8. Twelve gauges were installed to monitor strains in eight stringers and six were installed to monitor strains in the first two floor beams.

4.2/ Trusses

A total of 37 strain gauges were installed on members of the south truss to monitor axial strains, as shown in Figure 9. One displacement transducer was attached at midspan to monitor vertical deflection. Only five strain gauges were installed on the north truss, as shown in Figure 10.

4.3/ Equipment and Accuracy

The instrument gauges were monitored using MTO's computer-based data acquisition system which stores all data on a floppy disk while simultaneously providing an on-site hard copy. The demountable strain gauges have an accuracy of about ±5 microstrain, while the electrical resistance gauges have an accuracy of about 1 or 2 microstrain(s). The displacement transducers are accurate to about ±0.14 mm. These approximated accuracies account for the installation procedures.

5/ Load Testing

Load testing of the bridge was performed on June 5, 1986, using only one of MTO's load testing vehicles. A maximum static load equal to 77 tonnes was successfully applied with no evidence of distress.

5.1/ Vehicle Loads

The bridge was tested using the Kenworth-King tractor-trailer combination, the dimensional configuration of which is given in Figure 11. The proposed test loads, in terms of tandem/axle weights are provided in Table 4.

5.2/ Vehicle Placement

The vehicle was placed so as to produce maximum responses at all critical instrumented sections. The vehicle was placed in three lines or lane positions as shown in Figure 12.

The critical longitudinal positions, or positions along the bridge in each line, were derived from the analytical techniques used in the preliminary evaluation to produce maximum responses under the test vehicle. The longitudinal positions are given in Figure 13. Only positions 1 to 6 were applied in lines 1 and 2. All positions were applied for line 3.

5.3/ Testing

Load testing was performed on June 5, 1986, under a cloudy sky with periods of rain. The rain tended to affect the operation of a number of gauges, particularly during the last load level. For each load level, the vehicle was moved through each line position in turn, stopping at each of the required longitudinal positions while the instruments were recorded. Zero offset readings were taken before and after each line had been run.

6/ Test Results

The following section presents the results of the load testing along with, where applicable, comparisons with the analytical models used in the preliminary evaluation.

6.1/ Steel Stringers

Figures 14 a, b and c, display some of the test results for the stringers for load levels 1, 3 and 5. The plots give the steel strains in the top and bottom flanges plotted on the same scale with negative indicating compression. In all cases the results indicate that some composite action is taking place between the stringers and the concrete deck. The degree of composite action appears to vary, particularly between the interior and exterior stringers. The latter makes it difficult to simulate an accurate response analytically for design and evaluation purposes.

The distribution pattern of strains across the width of the deck for each vehicle position appears to be unaffected by the level of load. Therefore the accuracy and repeatability of the instruments is confirmed. Figure 15 displays measured strains versus increasing load for the first interior stringers on each side of the deck. This figure indicates the stringer/deck system responded fairly linearly considering the possible experimental error, particularly with respect to the repeatability of vehicle placement.

Note that the tensile strains given in Figures 14 a, b and c, for line 3, where the vehicle was symmetrically placed about the centre line of the roadway, are themselves symmetrical. Therefore, if we consider that the degree of composite action was consistent for the interior and exterior stringers then the strains in the top flanges of those stringers where no gauges were installed, may be projected as shown in Figure 16. This may be viewed as a large assumption but it will allow us to estimate the total moment in the deck when we are evaluating the structural responses in Section 7.

6.2/ Steel Floor Beams

Figures 17 to 19 display measured strains in the floor beams under several vehicle positions and load levels. Figure 17 indicates that floor beam No.1 responded with varying degrees of composite action where the position of zero strain changed sharply with load level and changed to a much lesser degree with vehicle position. It is interesting to note that under load level 5 there is less change in the zero strain location with different vehicle positions. This implies that a high load

is required to establish a more consistent composite action. Figures 18 and 19 tend to bear this out as they each display a near consistent zero strain location for floor beam No. 2 while the overall magnitude of the strains are much higher than for floor beam No. 1.

Figure 20 shows the measured strains versus increasing load for both floor beams with the vehicle in line 3. Although the responses appear to be almost linear all the lines appear to curve upwards as the load reaches level 5. The latter indicates a possible stiffening action. This could be caused by increasing composite action or end restraint, although the latter is less likely as the truss would provide minimal support to restrain the ends of the beams.

6.3/ Steel Trusses

The test results are presented as member axial forces derived from the measured strains at a section as well as the strains themselves. Figures 21 to 35 and Figures 36 to 47 represent load levels 3 and 5, respectively, for the test vehicle moving through line position 3. These figures also display the results of the analytical model under the same loads and positions.

Member 1 was one of few members where the instrumentation was such that any bending effects could be isolated in order to obtain the true experimental axial component. However, the comparison with the predicted or analytical values, as given in Figures 21 and 36, display a difference at the maximum of about 25%. This would be on the conservative side if we were to use the analytical model as a design/evaluation tool. Although it is possible that the performance of the deck with the bottom chord, as will be discussed shortly, could have something to do with this discrepancy there is no strong evidence to support any reason at this time. In any case, this member was not identified as being critical in the preliminary evaluation.

The bottom chord responses are typified by Figures 23, 24, 38 and 39, where the experimental is far below the analytical. This performance has been observed many times in steel trusses as the deck system is known to participate with the bottom chord carrying tensile forces.

Member 21 displayed in Figures 25 and 40, is the only vertical member in the truss which is made up of two angles. Because of size and geometry it was not possible to place the instruments at the neutral axis such that any bending effects would be eliminated. Also, because of the lack of additional instrumentation on the other legs of the angles we cannot determine how much, if any, bending strain was being measured. It is necessary to consider these points when observing the

discrepancy between the analytical and the experimental. In addition, the floor beam is attached to this member on the inside. The effects of this are discussed below.

The remaining vertical steel members are all composed of two symmetrically placed channels such that the instruments could be placed to eliminate any in-plane bending effects. However, these members also have some interesting features which should be noted. The floor beams in the deck are connected to these members near the bottom chord using double angles and rivets connecting the webs of the channel and floor beams. Details of this connection were not obtained. However, the effects of this connection are demonstrated in Figure 26 where the experimental results are given for each gauge separately. The inside channel is receiving a much larger share of the load than the outside. Even with this unbalanced loading the average value still compares favourably with the analytical. The same applies to the remaining verticals where the average experimental results are given in Figures 27 to 29 and 41 to 42. These members were not identified as being critical in the preliminary evaluation.

All the remaining members of the truss are internal diagonals which are composed of two angles, the same as member 21. They were also instrumented in the same way and therefore the results are susceptible to the effects of in-plane bending. Even so, the comparison with the analytical results is still quite good with only several members needing some additional discussion.

Member 31 was identified as a critical member in the preliminary evaluation. The test results as presented in Figures 30 and 43, display some discrepancy when compared to the analytical. However, for this particular member the outside gauge was identified as being very suspect due to the large drift of the output between the time of loading and after the load was removed. Figure 43 displays the experimental result using only the inside gauge. The latter shows a marked increase in the member force with the values coming a little closer to the analytical.

Member 38 also displays a considerable difference between the experimental and analytical as shown in Figures 34 and 47. Again, one of the gauges was displaying considerable drift. However the other gauge was also drifting slightly and this may be the reason the comparison for this member is very poor. Since both gauges were suspect no further steps were taken.

In general the experimental versus analytical comparison given in these figures provides reasonable support to the validity of using the frame analysis as an analytical tool for this truss. In most cases the model appears to overestimate the forces which would lead to a conservative evaluation.

7/ Post-Testing Evaluation

Following load testing the bridge was re-evaluated for the OHBDC design loads.

7.1/ Steel Stringers

The stringer responses as presented in Section 6 appeared to be reasonably consistent under increasing load. Table 5 presents the experimental strains from the bottom flanges of the stringers for 3 load levels under longitudinal position 5 for all three lanes. The distribution of the steel strains across the width of the deck can be used to approximate the moment distribution for the stringers. By summing the strains from each stringer for one vehicle position and dividing the sum into the maximum strain we have the critical load fraction. The latter represents the fraction of the total section moment induced by the full vehicle that is carried by the maximum loaded stringer. Subsequently this load fraction can be related to the distribution width D_d by the following equation;

$$\frac{S}{D_d} = (\text{calculated load fraction}) \times 2$$

where S is the stringer spacing

The worst distribution width for each vehicle position was determined using the strains given in Table 5. The results are listed in Table 6. The distribution widths are reasonably consistent for each of the vehicle positions. The critical value was found to be under the eccentric vehicle position for the first interior stringer. In addition it should be noted that this value of 1.72 m is considerably less than the 2.09 m determined from the OHBDC [1] in Section 3. There are two factors which are believed to contribute to this difference between the experimental and design distribution values:

1. The deck system displays composite action between the slab and the stringers as discussed in Section 6. Since composite decks will display worse distribution characteristics than non-composite systems this may explain part of the reason the distribution appears to be more critical than that given by the OHBDC.

2. As described in Section 2 and noted in Figure 3 there was a gap between the concrete slab and the top of the exterior stringers throughout the deck. This gap exceeded 25 mm in some cases. Therefore, under loading, the deck would have to move down before it would engage the outside stringers. This would introduce more loading on the first interior stringers as is evidenced by the results displayed in Figures 14 a, b and c. This condition is believed to be an important factor in the poor distribution performance of the deck with high loading in the first interior stringers.

A recent report by Bakht [3] presented a useful method for approximating the total section moment across the width of a slab-on-girder bridge. This method utilized the measured strains in the top and bottom of the steel beams. However in order to use this procedure it is necessary to assume some compressive strains for three of the stringers which were not instrumented. Considering the consistency of the measured strains, as presented in Section 6, the needed values were determined assuming these stringers performed with the same degree of composite action as their counterparts symmetrically located on the other side to centre line. The resulting strain distributions are given in Figure 16 for load level 5.

The method proposed by Bakht determines the total moment for a partially or fully composite girder system. The method breaks down the computation into three parts as given in the following equation;

$$M = M_L + M_U + Na$$

Here M_L and M_u are the moments taken by the naked girder and deck sections respectively, bending about their own respective axes, and N is the axial force acting in opposite directions in the girder and slab to a lever arm a. The components of this equation can be found as follows:

$$M_L = \sigma_{fb} S_b$$

$$M_U = M_L(EI_S/EI_g)$$

$$Na = \sigma_a Aa$$

where σ_{fb} = flexural stress in the bottom flange of the steel stringer

S_b = section modulus of naked steel stringer with respect to bottom flange

EI_g = flexural rigidity of the naked steel stringer bending about its own neutral axis

EI_s = flexural rigidity of the slab bending about is own neutral axis

σ_a = axial stress in the naked steel stringer due to axial force N

A = cross-sectional area of naked steel stringer

a = lever arm of the couple due to axial force N (assume to be distance between neutral axes of stringer and slab as recommended by Bakht)

Using this method the total moment, M, was determined for load level 5, for the vehicle in longitudinal position 5 in each of the lane positions. The results are presented in Table 7 along with the theoretical moments assuming the deck system was first simply supported between floor beams and then fully continuous over the floor beams. The moments determined from the experimental data are very close to one another which supports somewhat the validity of the assumed compressive strains. As part of the calculations the worst distribution was determined using these experimental moments for each of the line positions. The latter are given in Table 7 and are very close to those using tensile strains given in Table 6. Assuming full continuity provides a moment of about 0.56 of the simply supported moment, while the calculated moments using the measured strains average about 0.63 of the theoretical simply supported moment.

In addition to the effects of continuity there is another factor which will influence the re-evaluation of the stringers. Normally the effects of the slab in a non-composite slab-on-girder bridge are ignored in the determination of the load carrying capacity because it is usually very small. However, due to the small size of these stringers the effects of the slab are significant, in fact the slab will carry at least 40% of the section moment.

In re-evaluating the stringers of this bridge we now have a number of considerations which need to be summarized.

1. The distribution width determined from the experimental results is over 20% smaller than assumed by design.

2. Continuity of the deck appears to reduce the total moment in the deck by more than 35% below the calculated value assuming simply supported stringers.

3. The concrete slab will carry at least 40% of the section moment.

4. The deck system is definitely displaying a considerable degree of composite action.

We cannot guarantee the degree of composite action that will be present beyond the maximum load that was applied during the test. Therefore we will begin by ignoring the effects in the initial re-evaluation. At some point there will be loss in composite action and this will improve the distribution, however we will assume the worst condition and initially take the distribution width to be 1.72 m.

Continuity of the deck will be taken as reducing the total simply supported moment by only 30% and the slab will be assumed to carry 40% of the section moment. Using these conservative assumptions we now get a capacity reduction factor F = 1.37 which still accounts for a 10% deterioration of the stringers. This value applies for all three evaluation levels given in the OHBDC.

Since this value of F exceeds 1.0 no additional investigation was considered necessary.

7.2/ Steel Floor Beams

The floor beam responses as presented in Section 6 indicated these members did not perform as consistently as the stringers. Under lower strains the degree of composite action changed as the load increased.

It is not feasible to calculate the total moment in the floor beams using the method applied in Section 7.1, as there were not enough floor beams instrumented.

In re-evaluating the floor beams there are a number of considerations which need to be summarized.

1. The floor beams are acting compositely with the concrete slab.

2. Using the effective slab width given in the OHBDC the slab will carry at least 12% of the section moment.

3. Continuity of the deck system in the longitudinal direction as described in Section 7.1 will increase the maximum design load assumed to be carried by a floor beam by at least 8%.

4. A 10% deterioration should be taken into account when calculating the moment capacity and 20% when calculating the shear capacity.

It is not possible to guarantee the degree of composite action beyond the level of load applied during the testing. Ignoring the composite action, the capacity reduction factors for the floor beams accounting for the above considerations would be F = 0.78 for moment and F = 0.81 for shear.

7.3/ Steel Trusses

An extensive post-testing evaluation was not performed for the steel trusses. The analytical model used in the preliminary evaluation was determined to be a reasonable approximation of the structural response of the truss as described in Section 6. If anything, the model provided conservative member responses for evaluation.

Reviewing the results of the preliminary evaluation given in Tables 1 and 2, we see that even ignoring the compressive capacity of members 38 and 39 does not change the response of member 31 which still governs the truss capacity.

At first glance the actual compressive force in member 31 appears to be only half of the predicted value as shown in Figure 30. However, as discussed in Section 6 and demonstrated in Figure 43 one of the strain gauges was not functioning properly. In addition, due to the instrumentation set-up, bending strains may be influencing the measured responses.

The member resistance for member 31 was re-investigated to evaluate the effects of changing the effective length of the member. Normally, and as applied in Section 3, the effective length factor for truss members is taken as 1.0. However member 31 is very flexible compared to the chord members into which it is framed. The end joints would provide some degree of end restraint which would reduce the effective length. We cannot justify a fixed end condition which would result in a K factor of 0.65 but a K = 0.9 should not be unreasonable. Using K = 0.9 the new

capacity reduction factors for member 31 would become F = 0.93, 0.86 and 0.86 for evaluation levels 1, 2 and 3, respectively.

This indicates that even a small increase in the assumed compressive capacity of member 31 makes a considerable difference. These values still account for a 5% deterioration of the member.

Using these new F factors for member 31, in conjunction with Table 2, the controlling values for the trusses become F = 0.93, 0.86 and 0.72.

7.4/ Summary

A summary of the scale down factors determined from the post testing evaluation are given in Table 8 along with the required posting restrictions. These indicate that the floor beams will govern the posting of the bridge for all three evaluation levels. Therefore, the recommended posting would be 18, 31 and 33 tonnes which is a marked improvement over the initial recommendation immediately following the load test of 15, 22 and 28 tonnes.

8/ Conclusions and Recommendations

The recommended posting for the bridge is 18, 31, and 33 tonnes for evaluation levels 1, 2, and 3, respectively. The final evaluation accounts for some deterioration in all the members considered. However, it is recommended that severely deteriorated members be repaired or replaced, particularly the exterior steel stringers and those members identified in Figures 5 and 6.

In addition, since shear in the floor beams appears to be a serious consideration any deterioration at the ends should be repaired. The latter is a concern since there is no benefit in shear from composite action whereas we know the flexural strength has some reserve in this regard.

The concrete slab should also be repaired or replaced due to its actively deteriorating condition.

The live load factor used in the evaluation was 1.25 and therefore this bridge will require re-evaluation within 5 years. However, the above recommended postings go hand-in-hand with immediate repairs to the members identified above.

References

[1] "Ontario Highway Bridge Design Code," Ontario Ministry of Transportation and Communications, Downsview, Ontario, 1983.

[2] "ICES STRUDL II - Structural Design Language," Massachusetts Institute of Technology, Cambridge, Mass., 1972.

[3] Bakht, B., "Testing of an Old Short Span Slab-on-Girder Bridge", Ontario Ministry of Transportation, SRR-88-01, 1988.

Member Number (See Figure 7)	Factored Member Resistance (kN)	Factored Dead Load (kN)	Maximum Factored Live Load OHBDC (kN)			Scale-Down Factor F OHBDC		
			Level I	Level II	Level III	Level I	Level II	Level III
1	-1668	-431	-644*	-935	-1104	1.92	1.32	1.12
5	-1679	-488	-478*	-694	-828	2.49	1.72	1.44
11	637	282	286*	414	489	1.24	0.86	0.72
13	935	379	380*	547	662	1.46	1.02	0.84
14	1031	447	431*	626	755	1.36	0.93	0.77
15	1128	478	459*	666	800	1.42	0.98	0.81
21	360	71.2	294	294	294	0.98	0.98	0.98
22	713	-42.7 †	187	202	202	4.04	3.74	3.74
	-534	-49.4	-137	-197	-219	3.52	2.46	2.21
30	459	157	201	292	331	1.50	1.04	0.92
	-68.9	135	-105	-105	-105	1.95	1.95	1.95
31	360	108	190	273	305	1.32	0.92	0.83
	0	93	-139	-151	-151	0.67	0.62	0.62
32	321	56.5	128	172	193	2.07	1.54	1.37
	-86.3	48.5	-65.4	-80.5	-81.8	2.06	1.67	1.65
33	272	29.8	117	153	170	2.07	1.59	1.42
	-69.4	25.8	-66.7	-94.3	-99.6	1.42	1.01	0.95
38	197	-20.9	85.0	106	106	2.57	2.05	2.09
	-42.3	-24	-80.5	-118	-134	0.23	0.15	0.14
39	197	-6.67	93.0	116	127	2.20	1.77	1.61
	-42.3	-7.56	-80.5	-117	-126	0.43	0.30	0.28

* Lane load governs
† Dead load factors of 1.0 are used when response reduces live load effect

Table 1/ Preliminary OHBDC Evaluation, Truss, Member Axial Forces

Member Number (See Figure 7)	Factored Member Resistance (kN)	Factored Dead Load (kN)	Maximum Factored Live Load OHBDC (kN)			Scale-Down Factor F OHBDC		
			Level I	Level II	Level III	Level I	Level II	Level III
1	-1668	-431	-644*	-935	-1104	1.92	1.32	1.12
5	-1679	-488	-478*	-694	-828	2.49	1.72	1.44
11	637	282	286*	414	489	1.24	0.86	0.72
13	935	379	380*	547	662	1.46	1.02	0.84
14	1031	447	431*	626	755	1.36	0.93	0.77
15	1128	478	459*	666	800	1.42	0.98	0.81
21	360	71.2	294	294	294	0.98	0.98	0.98
22	713	-42.7†	187	202	202	4.04	3.74	3.74
	-534	-49.4	-137	-197	-219	3.52	2.46	2.21
30	459	157	201	292	331	1.50	1.04	0.92
	-68.9	135	-105	-105	-105	1.95	1.95	1.95
31	360	108	190	273	305	1.32	0.92	0.83
	0	93	-139	-151	-151	0.67	0.62	0.62
32	321	56.5	169	246	283	1.43	**0.98**	**0.86**
	-86.3	48.5	-65.4	-80.5	-81.8	2.06	1.67	1.65
33	272	29.8	194	266	293	1.21	**0.88**	**0.80**
	-69.4	25.8	-66.7	-94.3	-99.6	1.42	1.01	0.95

* Lane load governs
† Dead load factors of 1.0 are used when response reduces live load effect

Table 2/ OHBDC Re-evaluation, with Members 38 and 39 Removed, Member Axial Forces

Minimum Scale-Down Factor, F

OHBDC Evaluation Levels

	I	II	III
Truss	0.67	0.62	0.62
Stringers	0.70	0.70	0.70
Floor beams	0.74	0.74	0.74

Table 3/ Summary of Preliminary Scale-Down Factors

Load Level	Rear Tandem (kN) Axle Nos. 4 & 5	Front Tandem (kN) Axle Nos. 2 & 3	Steering (kN) Axle No. 1	Total (kN)
1	130	134	46.9	311
2	180	177	47.9	405
3	223	225	50.4	498
4	268	271	52.7	592
5	316	314	54.9	685

Table 4/ Test Vehicle Loading (kN)

Load Level	Line Position	STEEL STRAINS FROM SOUTH TO NORTH SIDE					
		1	2	3	4	5	6
1	1	131	128	91	77	78	30
1	2	104	119	88	83	94	39
1	3	81	98	86	86	108	50
3	1	202	219	149	131	139	54
3	2	167	200	148	139	159	64
3	3	138	184	152	149	195	90
5	1	285	326	220	195	208	77
5	2	231	297	220	208	241	92
5	3	188	272	223	221	292	130

Table 5/ Experimental Strains in Steel Stringers (Longintudinal Position 5)

Load Level	Line Position	Sum of Strains*	Maximum Load Fraction	Experimental Distribution Width (m)
1	1	497	0.257	1.77
1	2	494	0.241	1.89
1	3	478	0.226	2.02
3	1	834	0.263	1.74
3	2	823	0.243	1.88
3	3	855	0.228	2.00
5	1	1226	0.266	1.72
5	2	1213	0.245	1.87
5	3	1252	0.233	1.96

*Exterior Stringer Strains adjusted to reflect smaller section size

Table 6/ Experimental Distribution Widths for Stringers (Longitudinal Position 5)

LINE POSITION	TOTAL SECTION MOMENT IN KN-M			
	Experimental	Predicted Moment		Distribution Width (m)
		Simply Supported	*Contin.*	
1	1577	2488	1390	1.77
2	1550	2488	1390	1.92
3	1588	2488	1390	2.02

Table 7/ Total Section Moments for Stringers

	Minimum Scale-Down Factor, F OHBDC Evaluation Levels			Required Posting (tonnes)		
	I	II	III	I	II	III
Truss	0.93	0.86	0.72	23	37	40
Stringers	1.37	1.37	1.37	NO POSTING REQUIRED		
Floor beams	0.78	0.78	0.78	18	31	33

Table 8/ Summary of Final Scale-Down Factors and Postings

Figure 1/ Glen Allan Bridge, Site No. 35-240

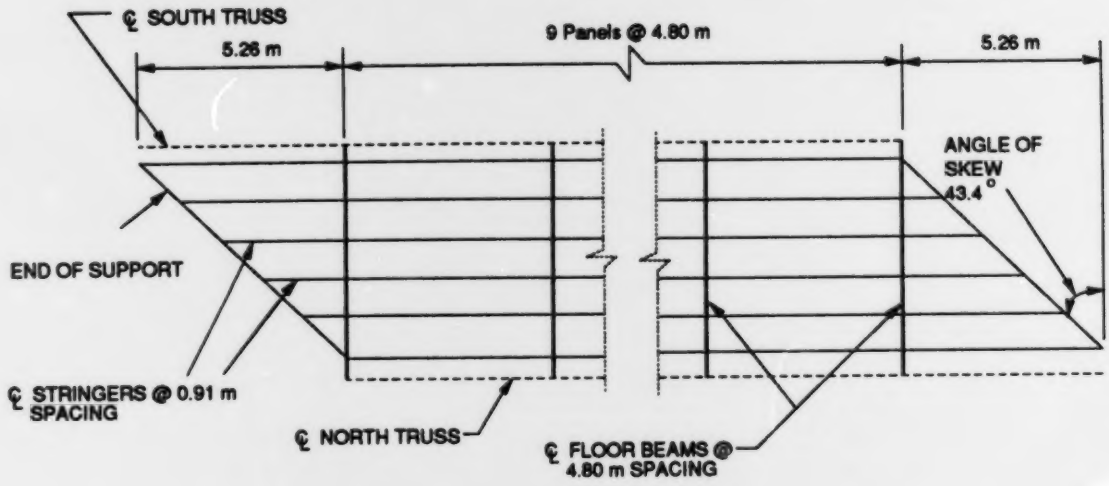
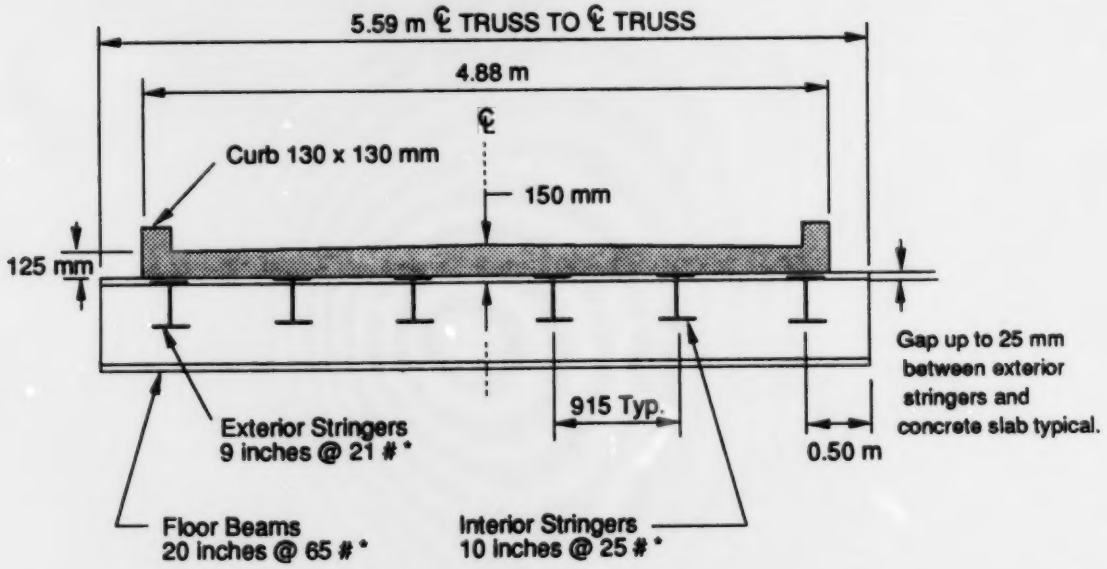
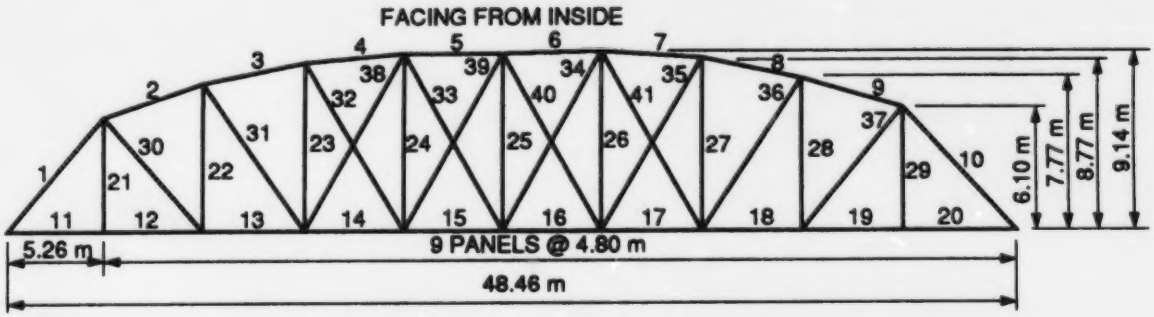


Figure 2/ Deck Support Plan


5.59 m ℄ TRUSS TO ℄ TRUSS
4.88 m
℄
Curb 130 x 130 mm
150 mm
125 mm
Gap up to 25 mm between exterior stringers and concrete slab typical.
915 Typ.
0.50 m
Exterior Stringers 9 inches @ 21 # *
Floor Beams 20 inches @ 65 # *
Interior Stringers 10 inches @ 25 # *

*- American Standard beams, no metric equivalents
- Assumed beam size from measurements and drawing

Figure 3/ Deck Cross-Section



Members 1 & 10

2 Channel C250 x 37 (C10"x 25 #)
& 1 Plate 406 x 4.5 mm

Members 2 to 9

2 Channels C250 x 30 (C10"x 20 #)
& 1 Plate 406 x 14.3 mm

Members 11,12,19,20

2 Angles 152 x 102 x 9.5 mm

Members 13 & 18

2 Angles 152 x 102 x 14.3 mm

Members 14 & 17

2 Angles 152 x 102 x 15.9 mm

Members 15 & 16

2 Angles 152 x 102 x 17.5 mm

Members 21 & 29

2 Angles 76 x 76 x 9.5 mm

Members 22 to 28

2 Channels C200 x 21
(C8"x 13.75 #)

Members 30 & 37

2 Angles 89 x 76 x 11.1 mm

Members 31 & 36

2 Angles 76 x 76 x 9.5 mm

Members 32 & 35

2 Angles 76 x 63.5 x 9.5 mm

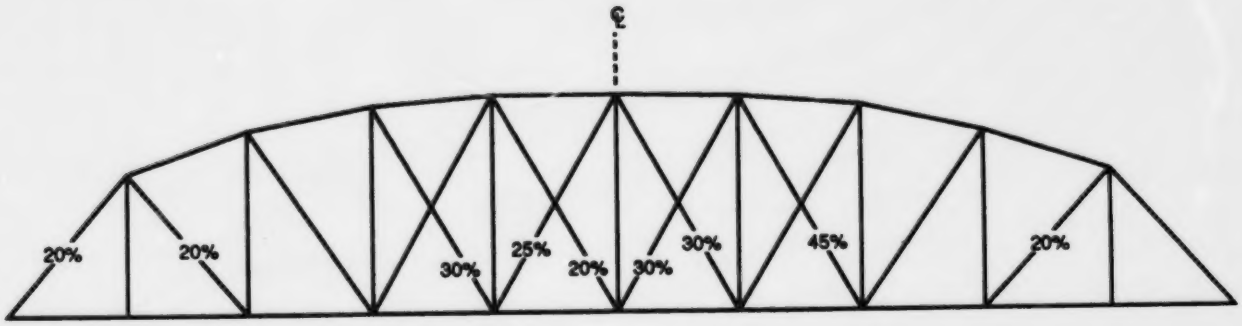
Members 33 & 34

2 Angles 76 x 63.5 x 7.9 mm

Members 38 to 41

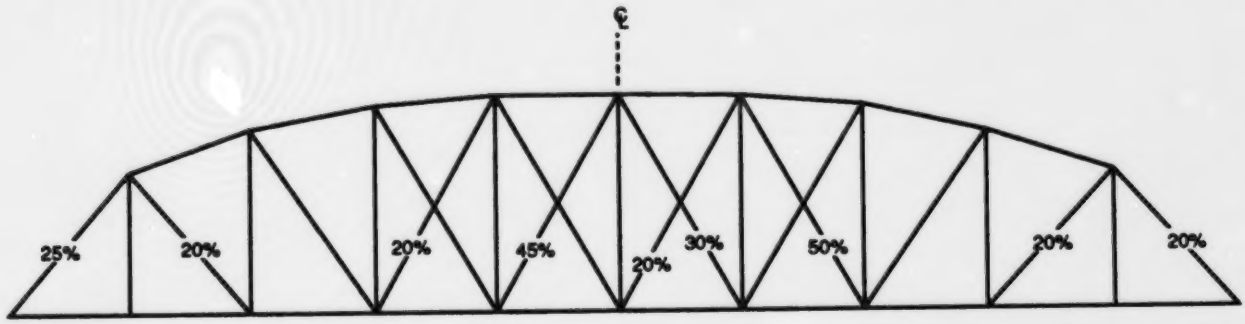
2 Angles 63.5 x 63.5 x 7.9 mm

Figure 4/ Member Size



ALL MEMBERS ASSUMED 15% SECTION LOSS UNLESS NOTED OTHERWISE

Figure 5/ Assumed Member Deterioration, North Truss (Facing South)



ALL MEMBERS ASSUMED 15% SECTION LOSS UNLESS NOTED OTHERWISE

Figure 6/ Assumed Member Deterioration, South Truss (Facing North)

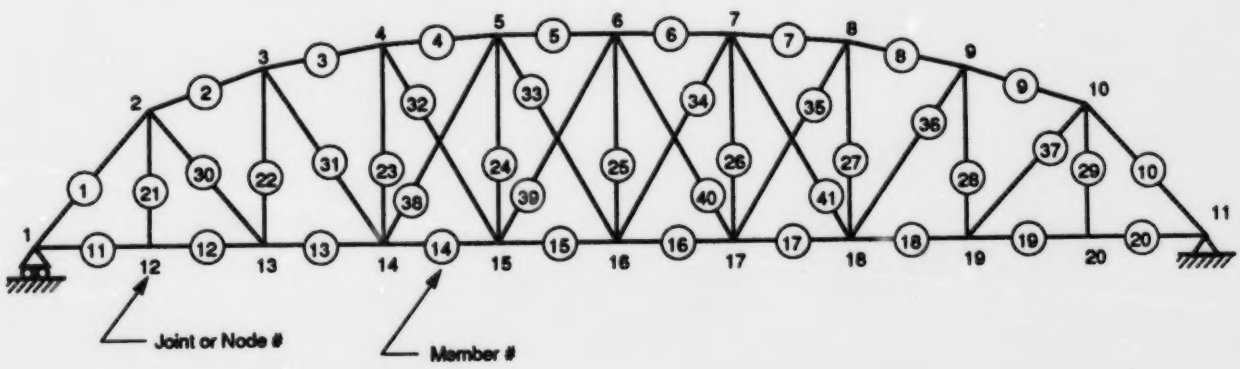
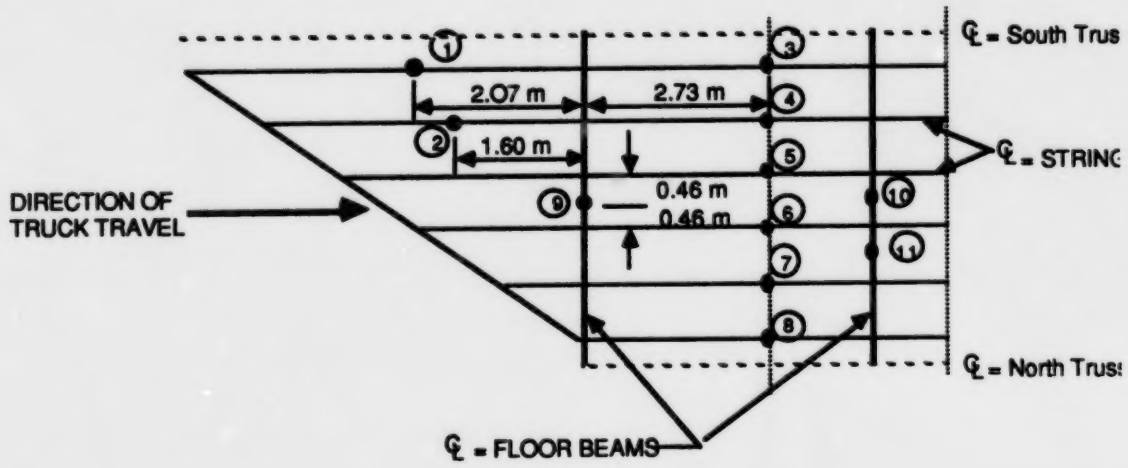


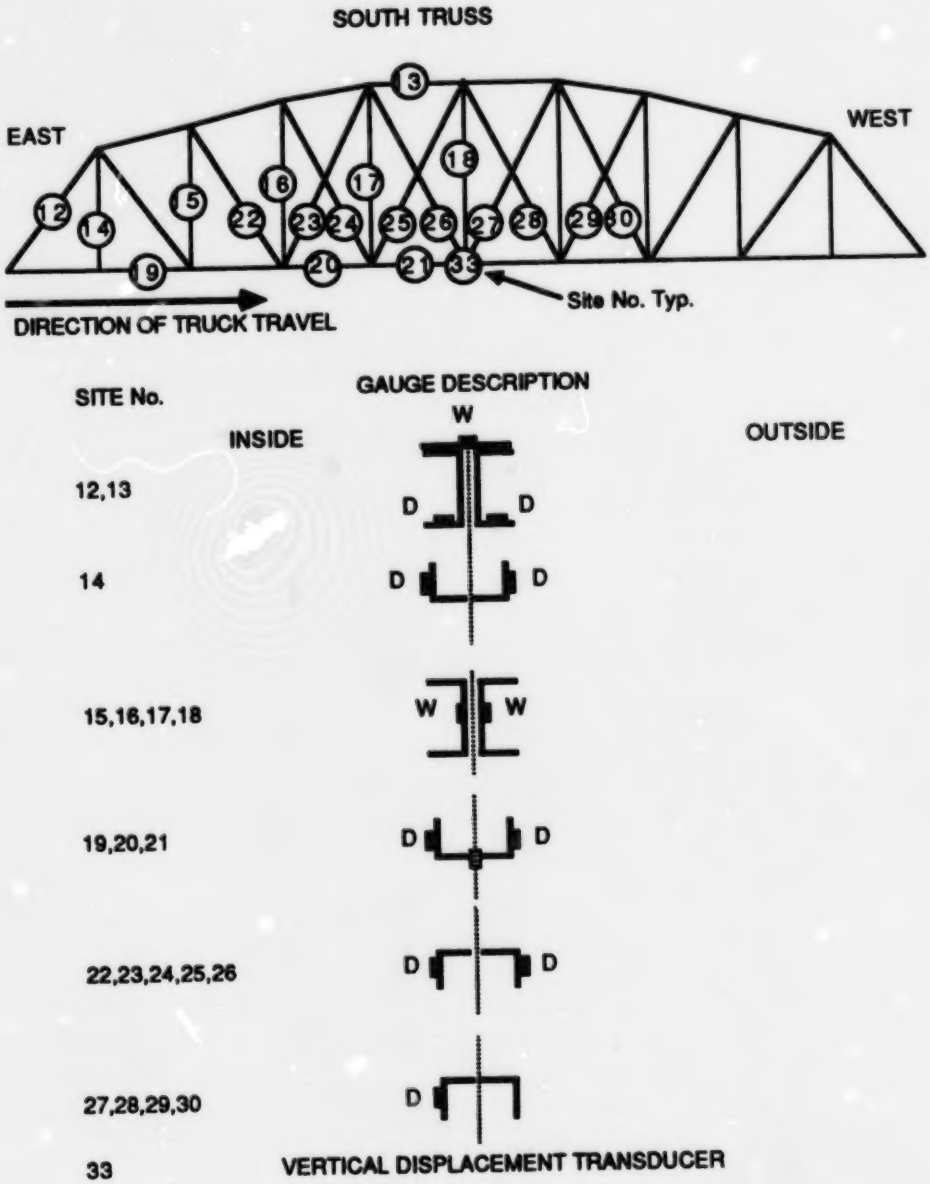
Figure 7/ Schematic Representation of Truss for Frame Analysis



Site No.	Gauge Description
1,3,4,5	W, D
2,6,7,8	D
9,10,11	W, D

W - Weldable Strain Gauge
D - Demountable Strain Gauge

Figure 8/ Bridge Deck Instrumentation



W - Weldable strain gauge
D - Demountable strain gauge

Figure 9/ South Truss Instrumentation

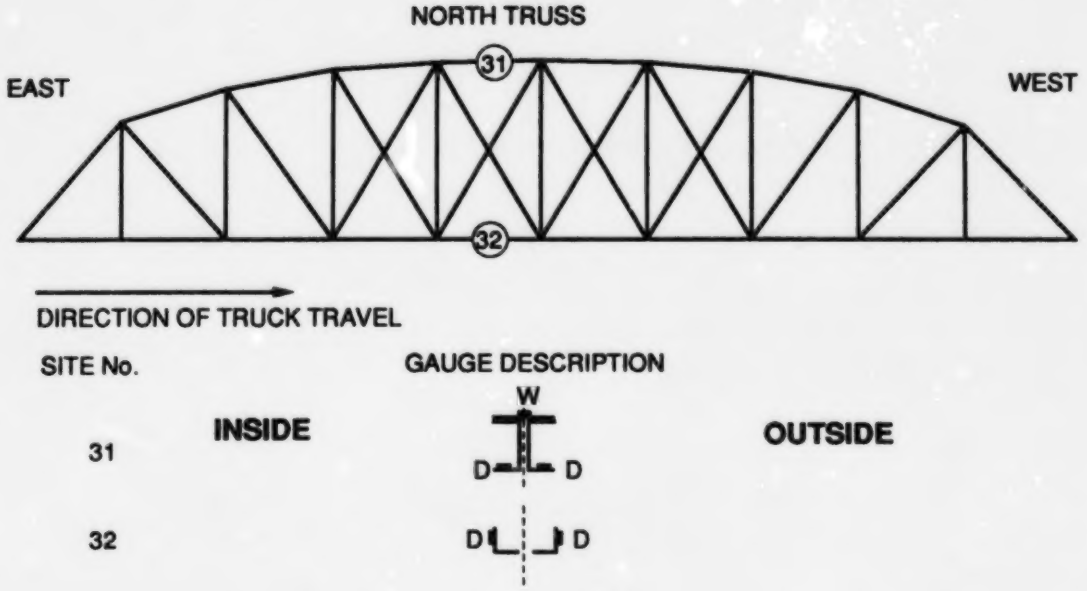


Figure 10/ North Truss Instrumentation

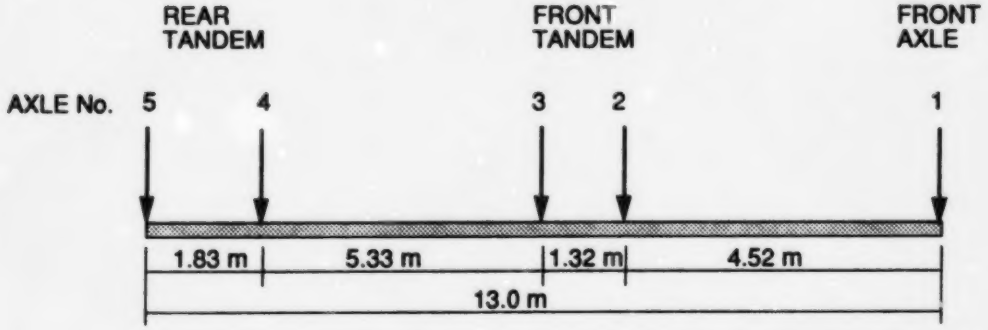
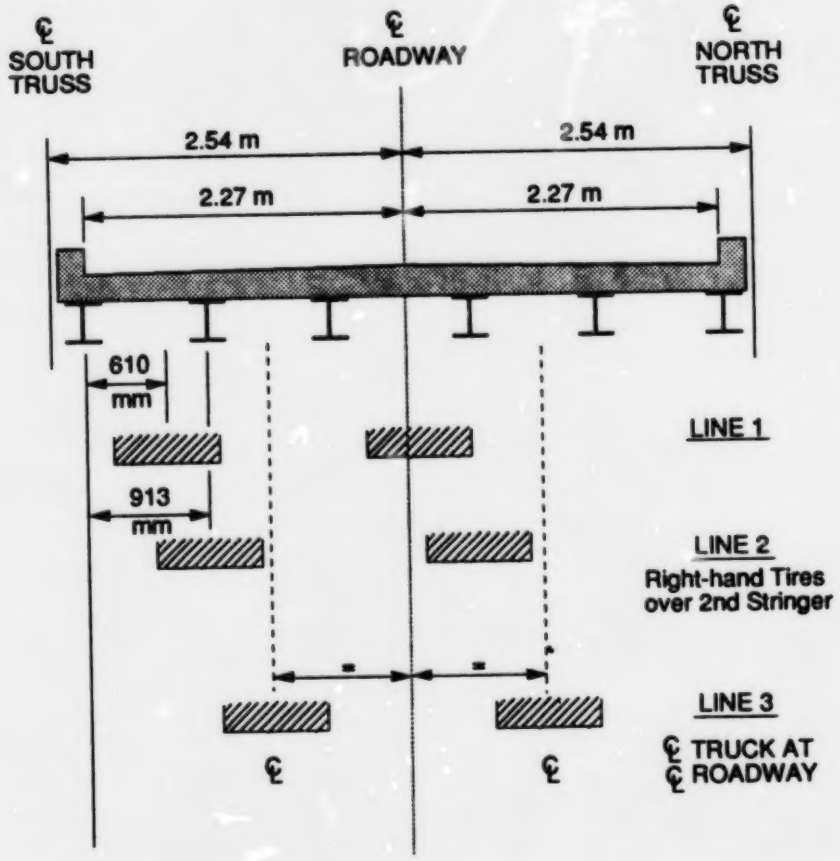


Figure 11/ Axle Configuration of Test Vehicle "Kenworth"



NOTE: Truck travelling from east to west

Figure 12/ Test Vehicle Placement, Line/Lane Positions

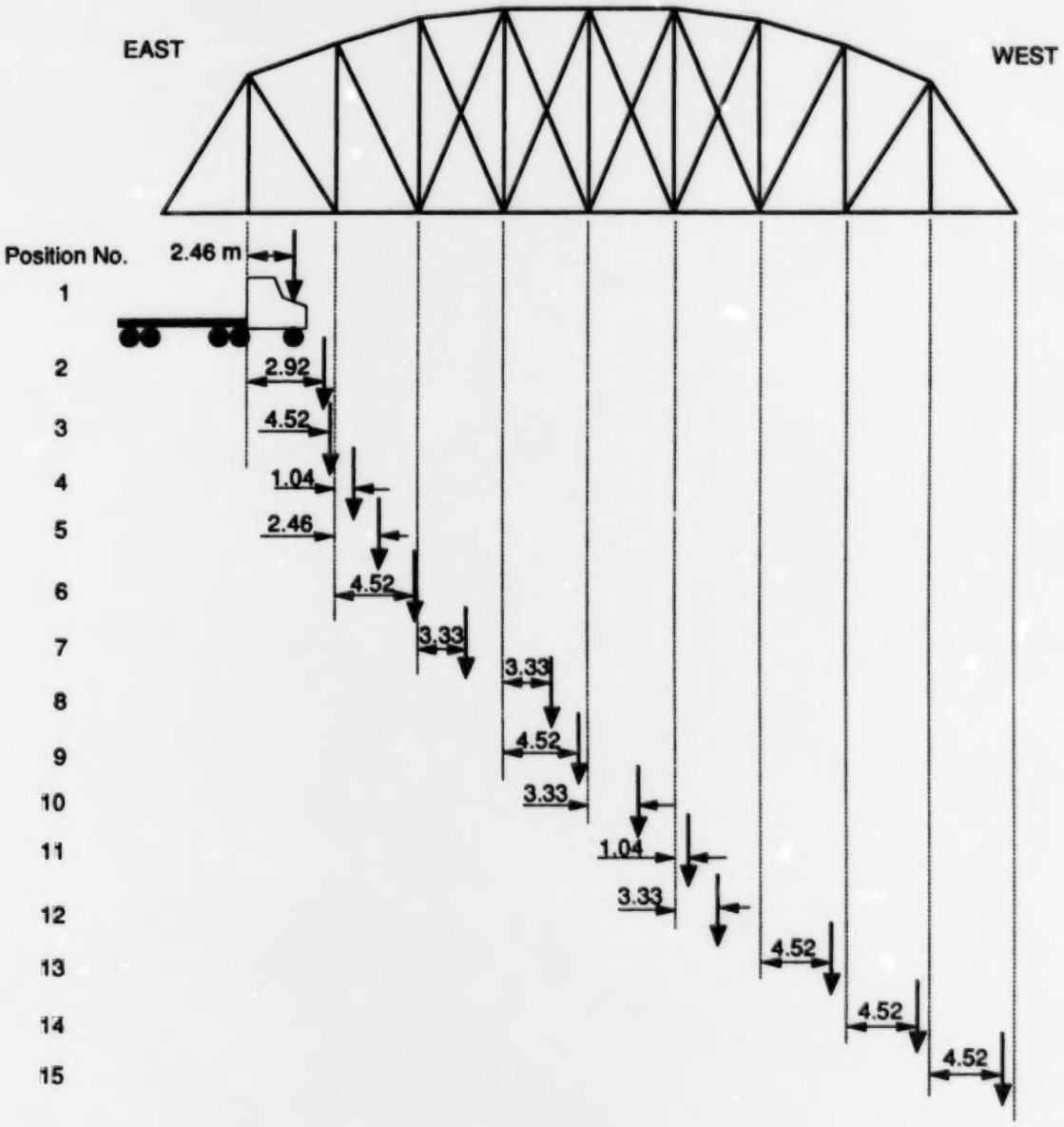


Figure 13/ Longitudinal Load Positions

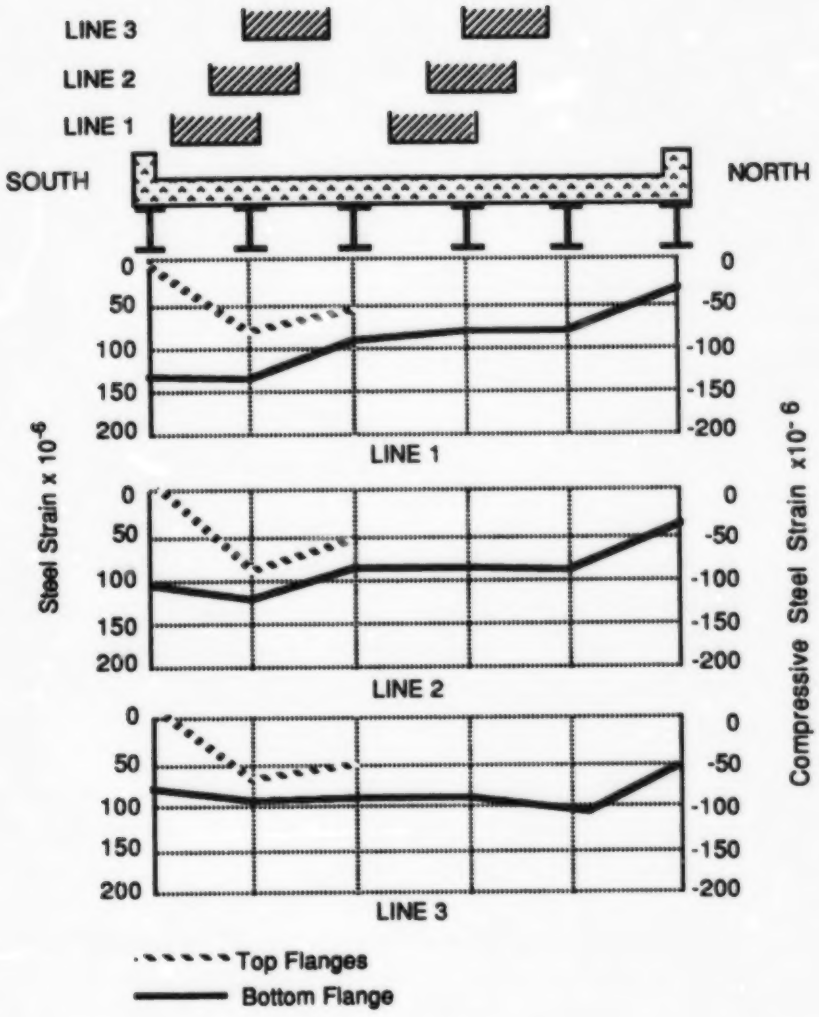


Figure 14a/ Test Results, Steel Stringers, Transverse Position 5, Load Level 1

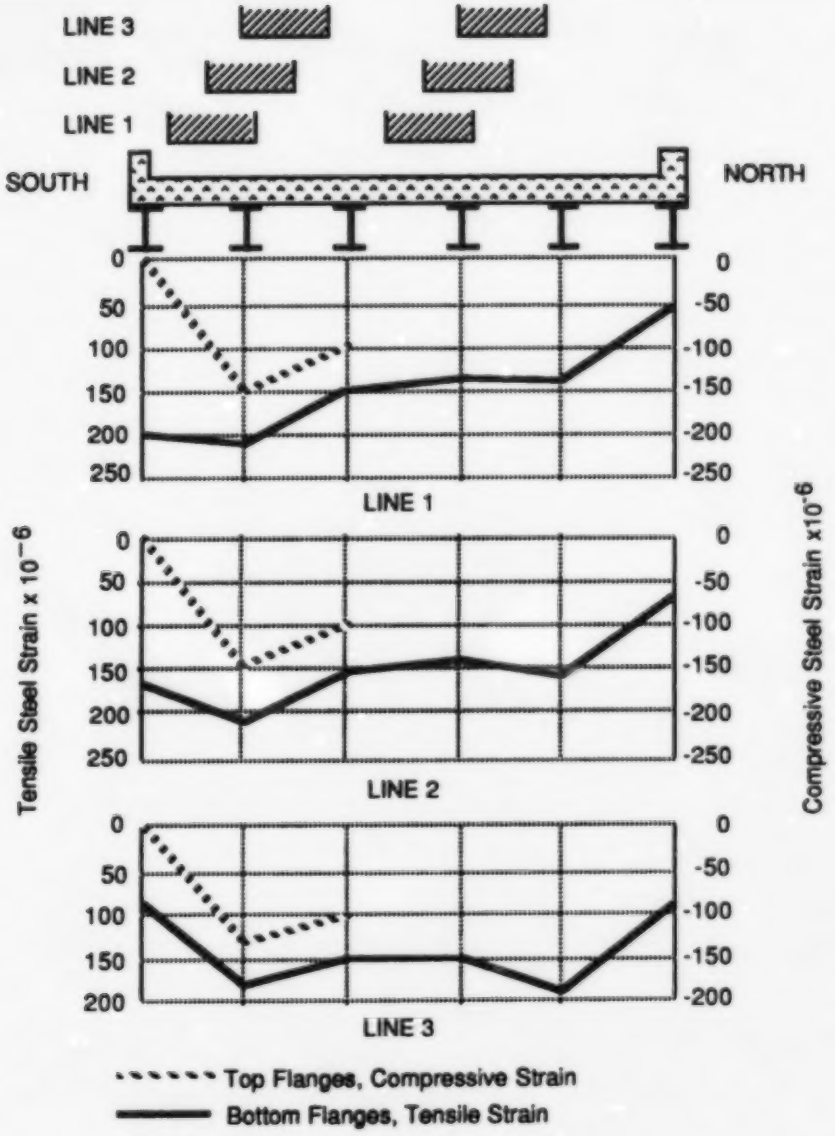


Figure 14b/ Test Results, Steel Stringers, Longitudinal Position 5, Load Level 3

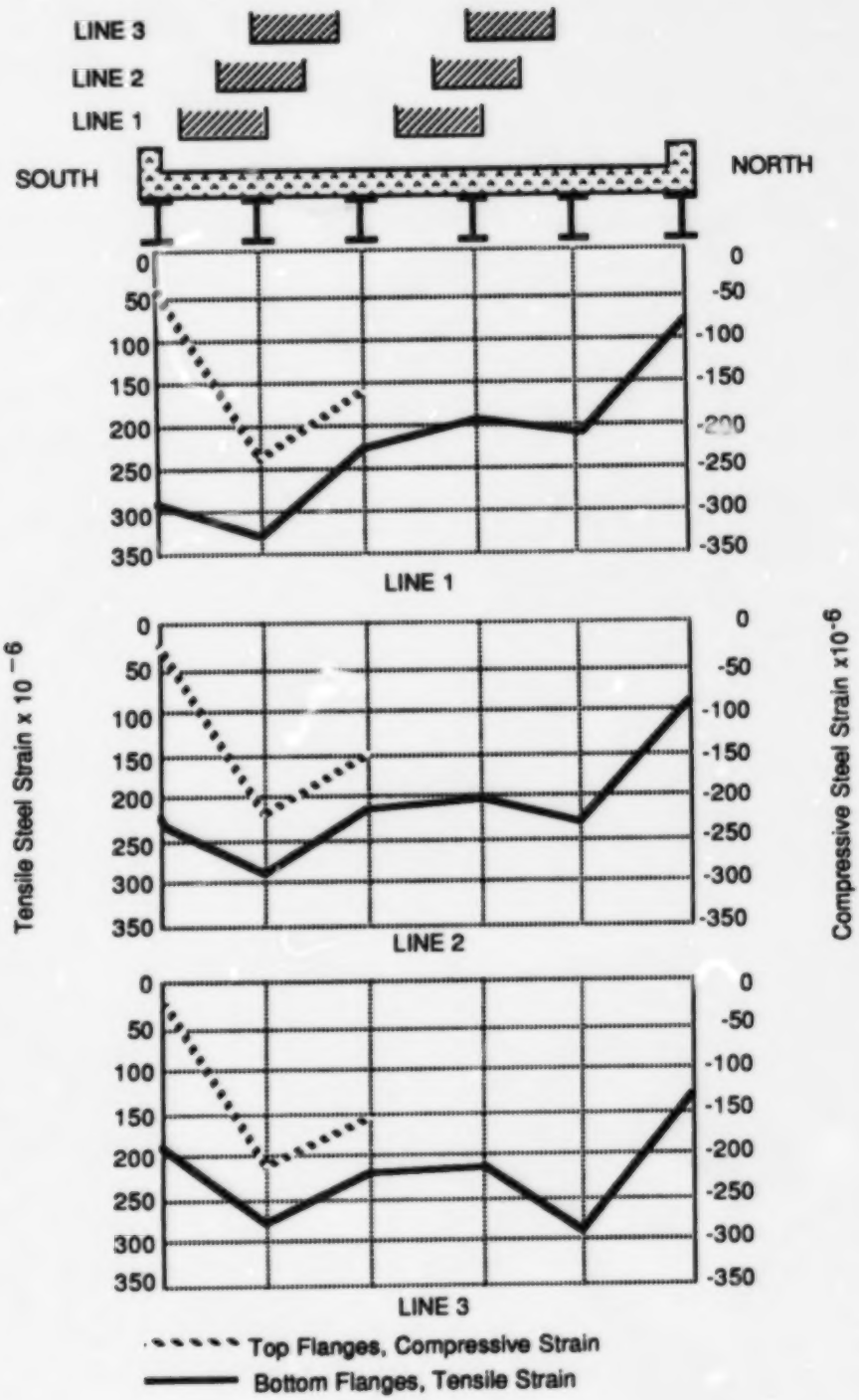


Figure 14c/ Test Results, Steel Stringers, Longitudinal Position 5, Load Level 5

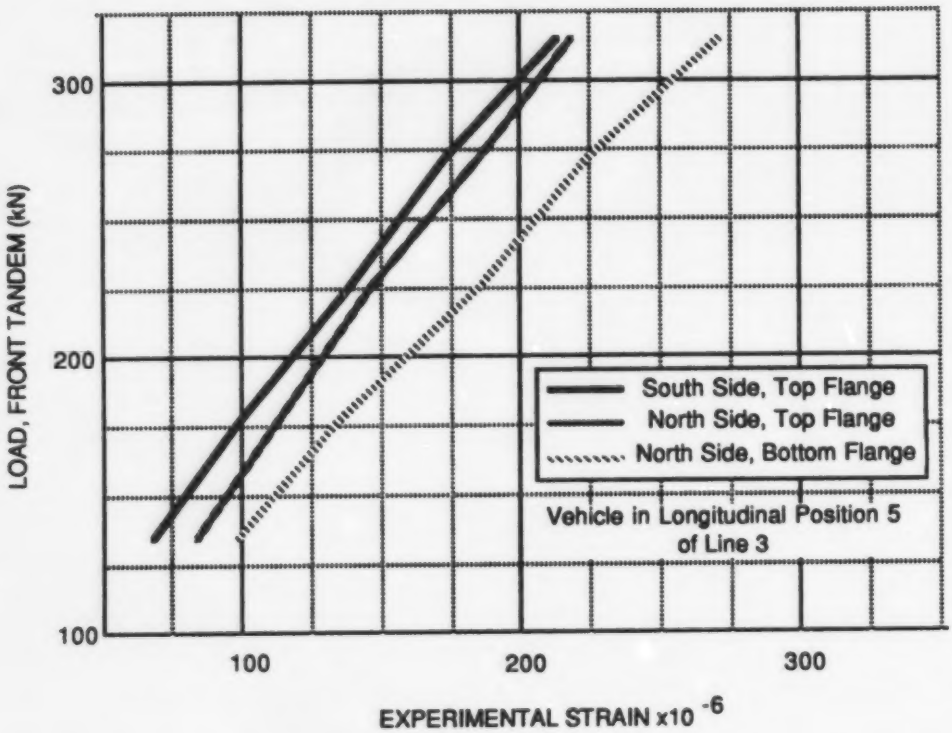


Figure 15/ Experimental Strain Versus Load for First Interior Stringers

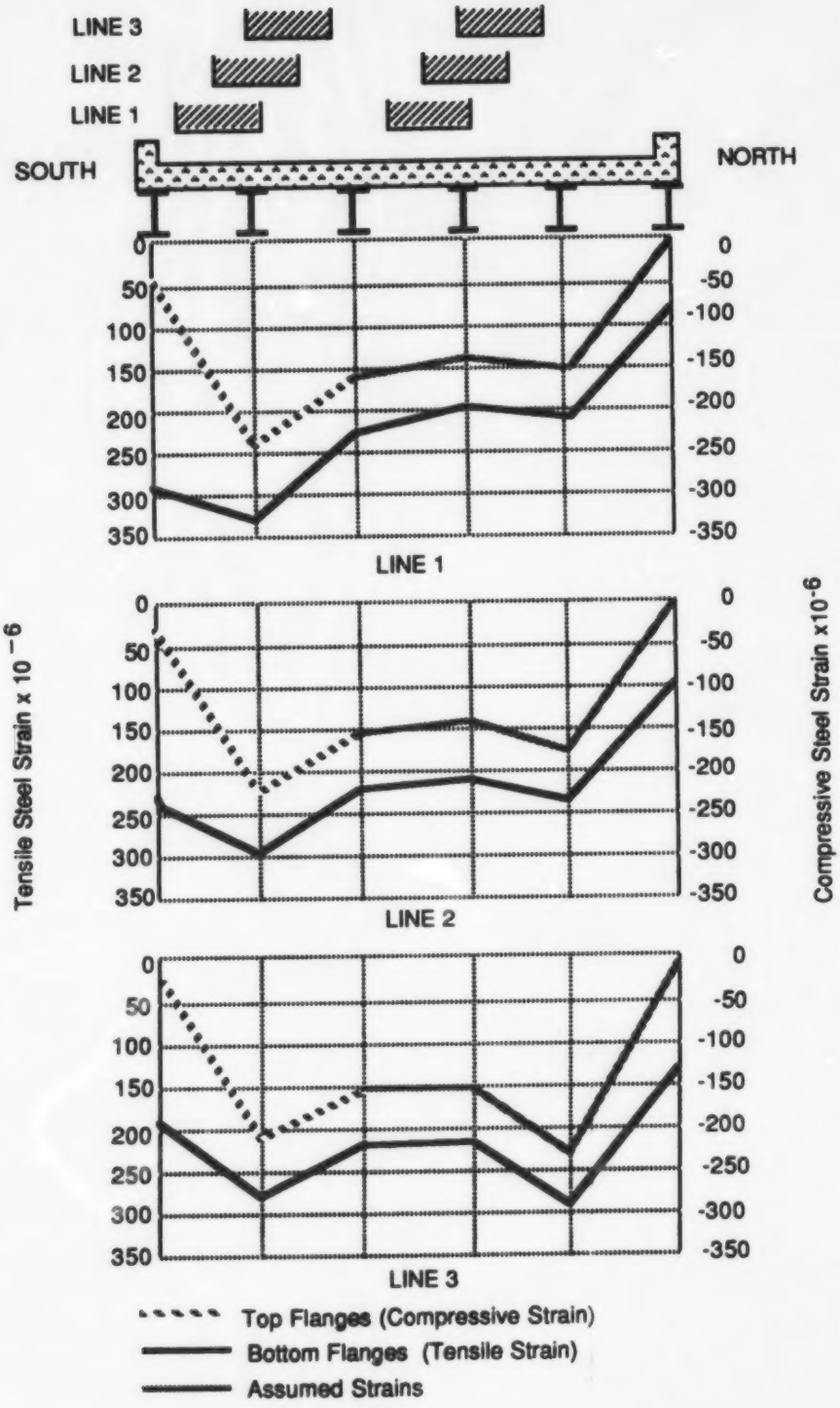


Figure 16/ Test Results, Steel Stringers, Longitudinal Position 5, Load Level 5

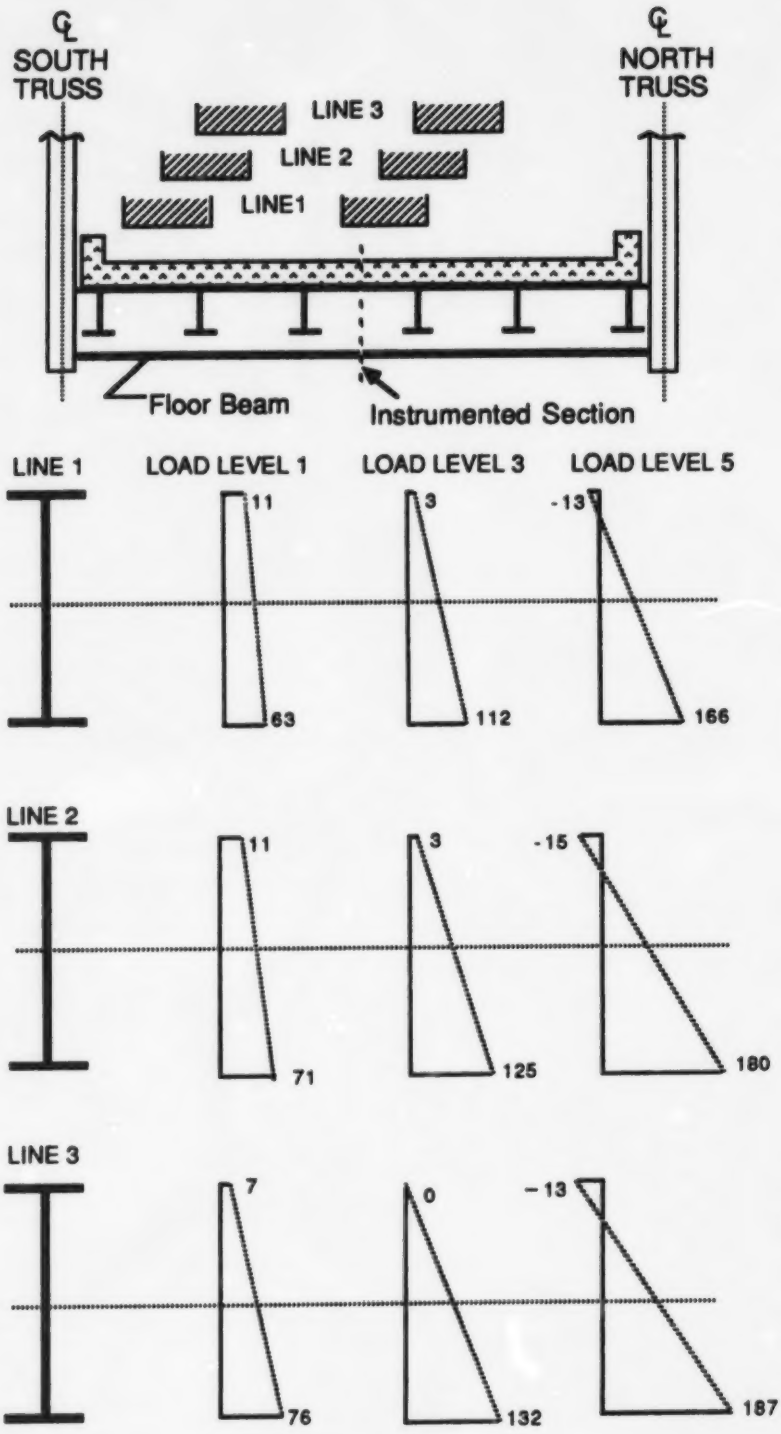


Figure 17/ Test Results, Floor Beam No. 1, Longitudinal Position (Microstrain)

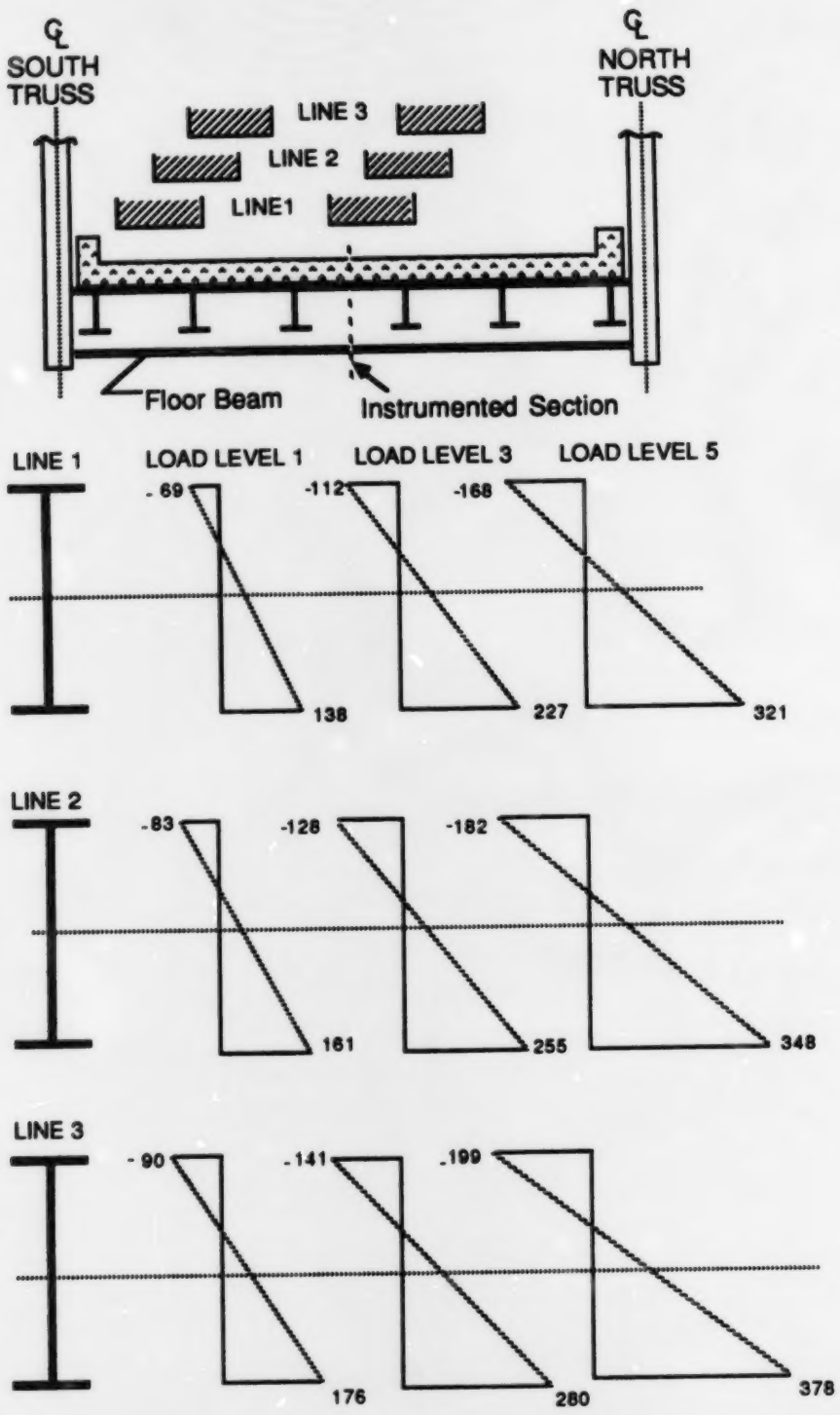


Figure 18/ Test Results, Floor Beam No. 2, Longitudinal Position 6 (Microstrain)

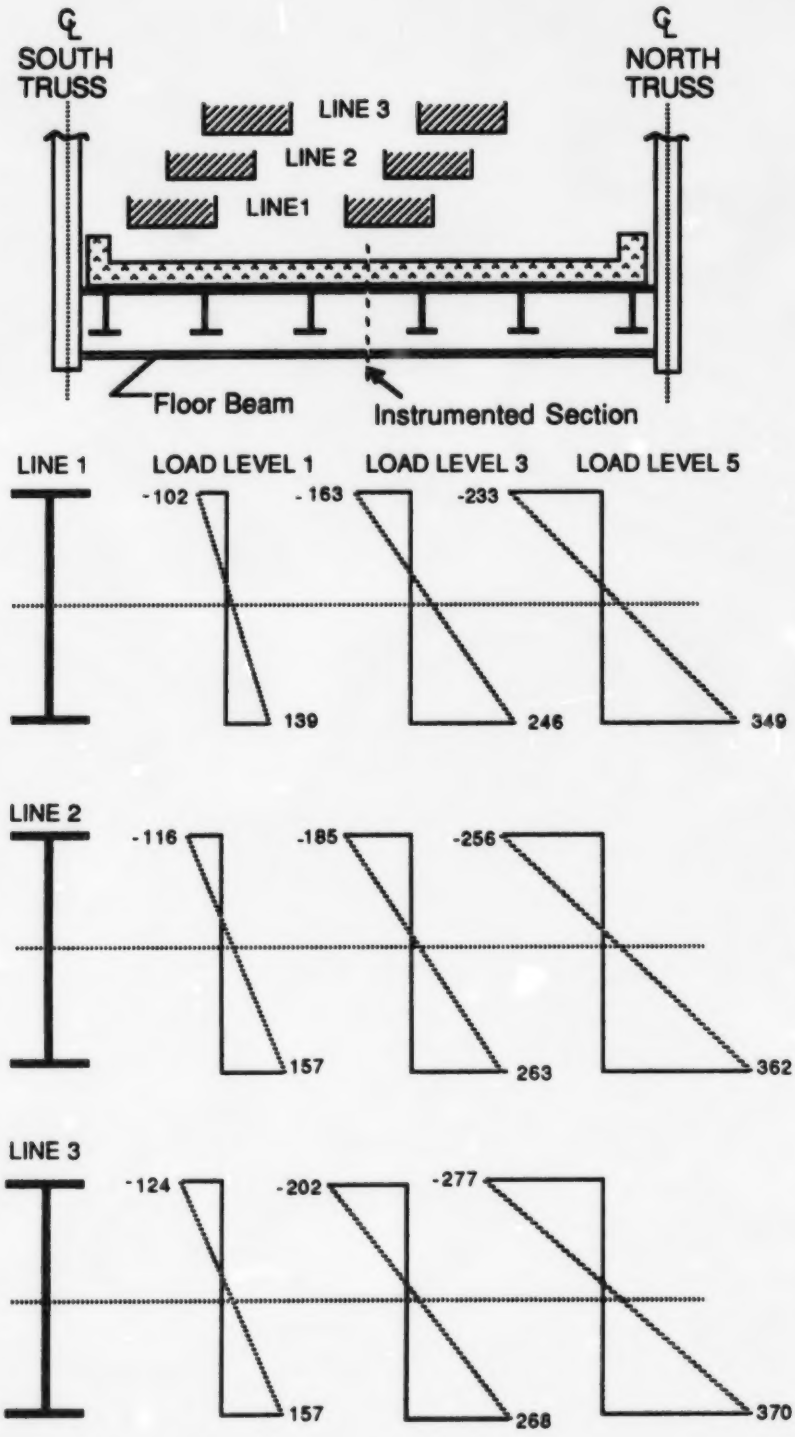


Figure 19/ Test Results, Floor Beam No. 2, Longitudinal Position 7 (Microstrain)

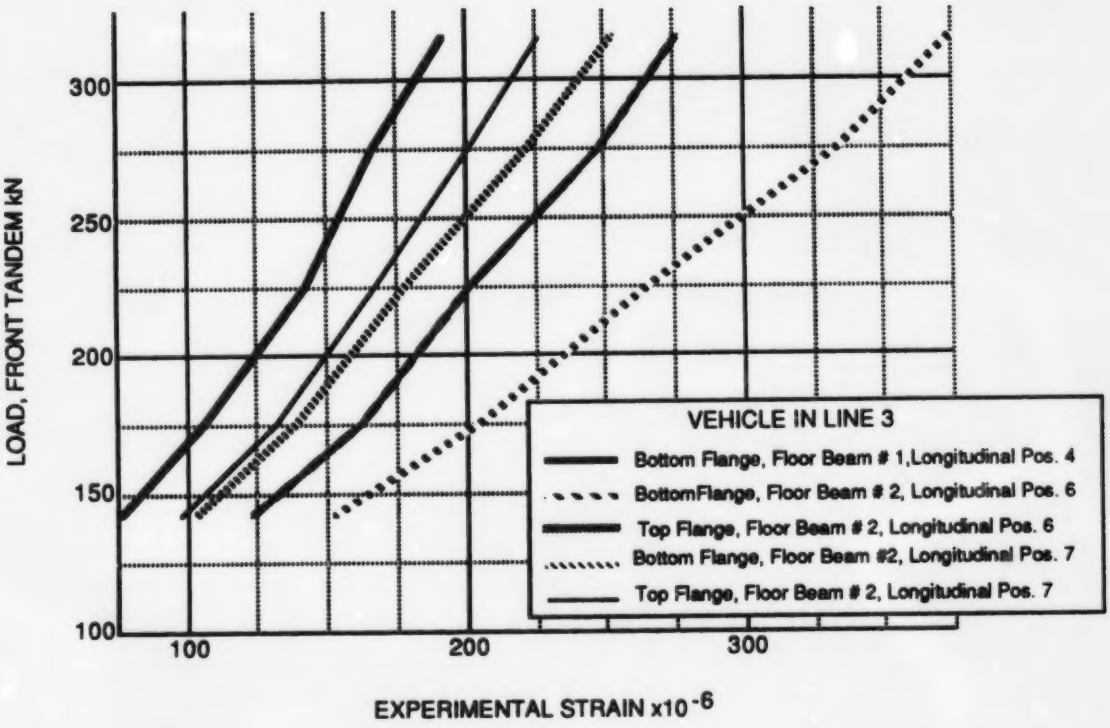


Figure 20/ Experimental Strain Versus Load in Floor Beams

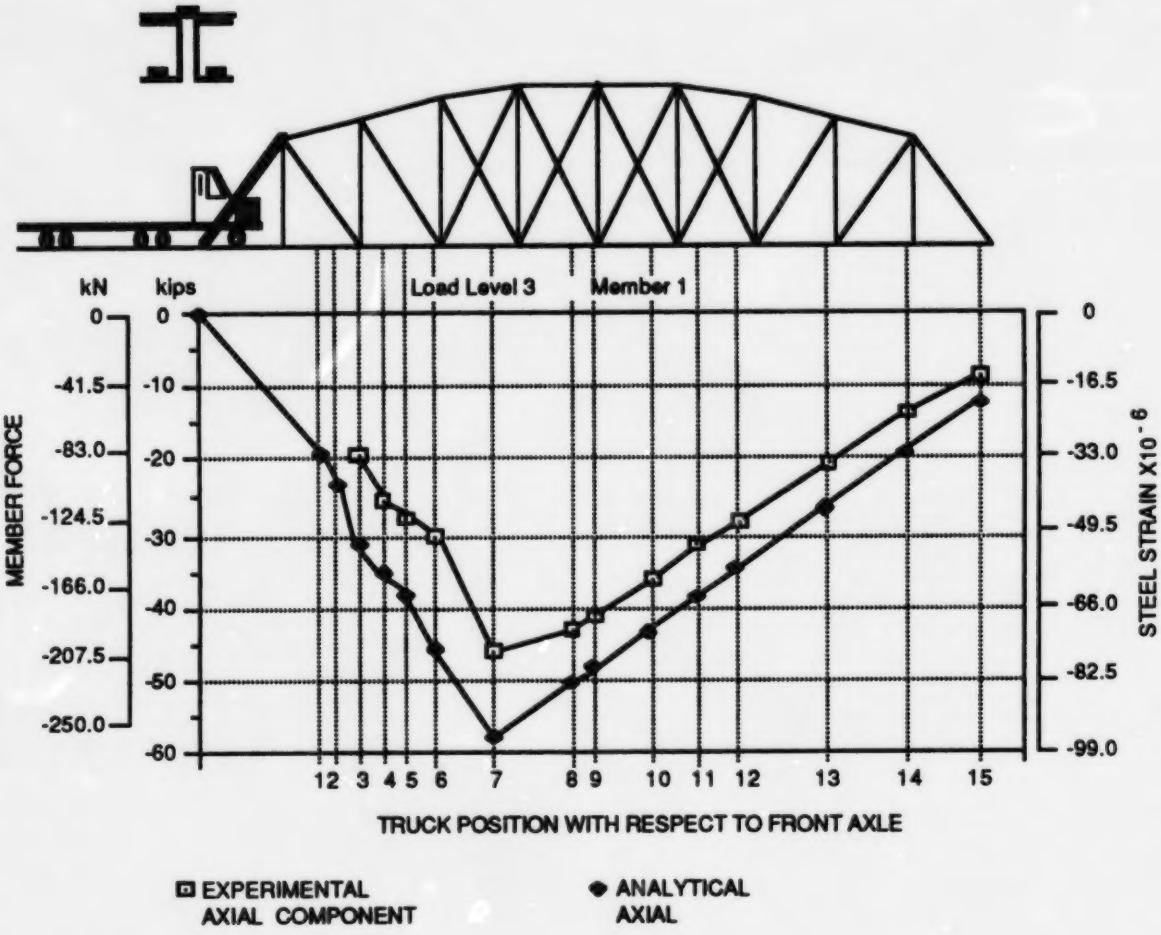


Figure 21/ Force Effects in Member 1

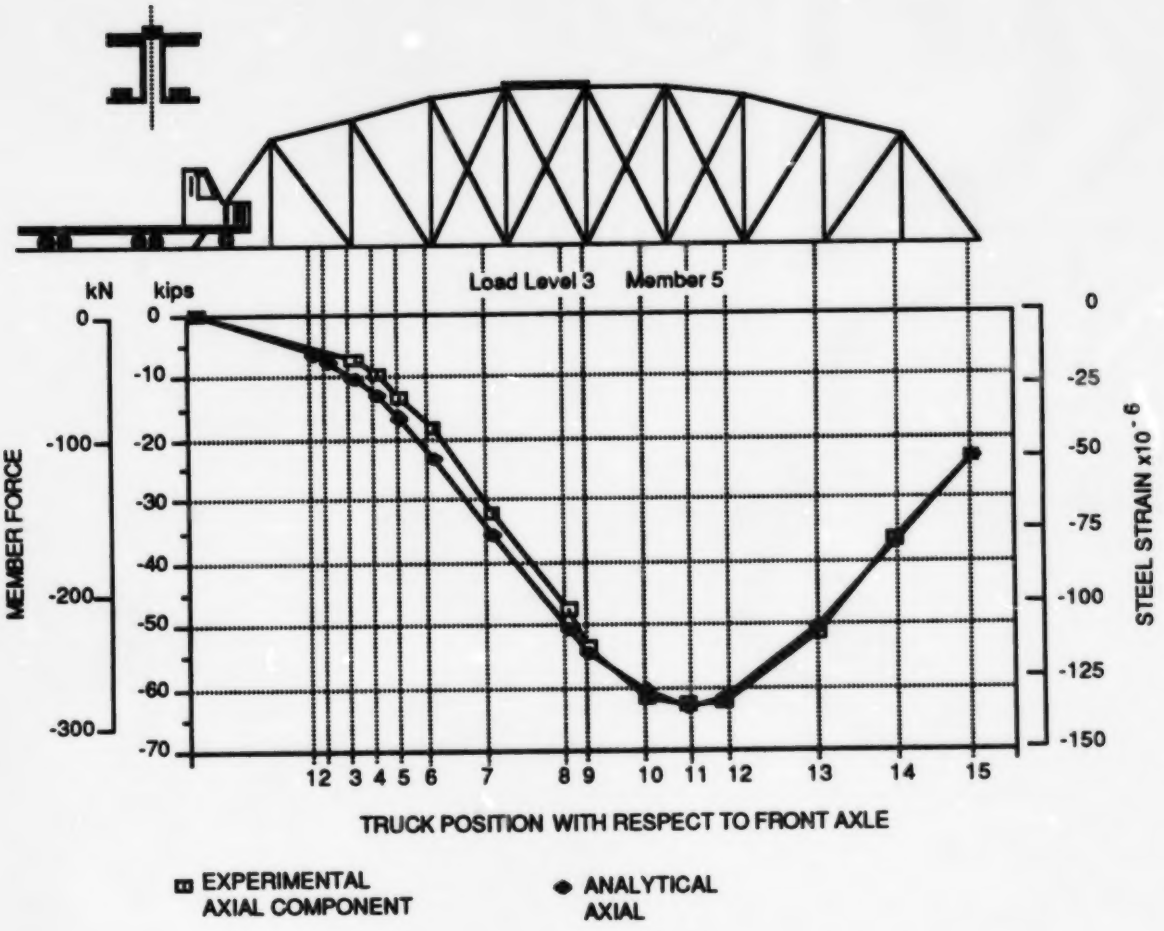


Figure 22/ Force Effects in Member 5

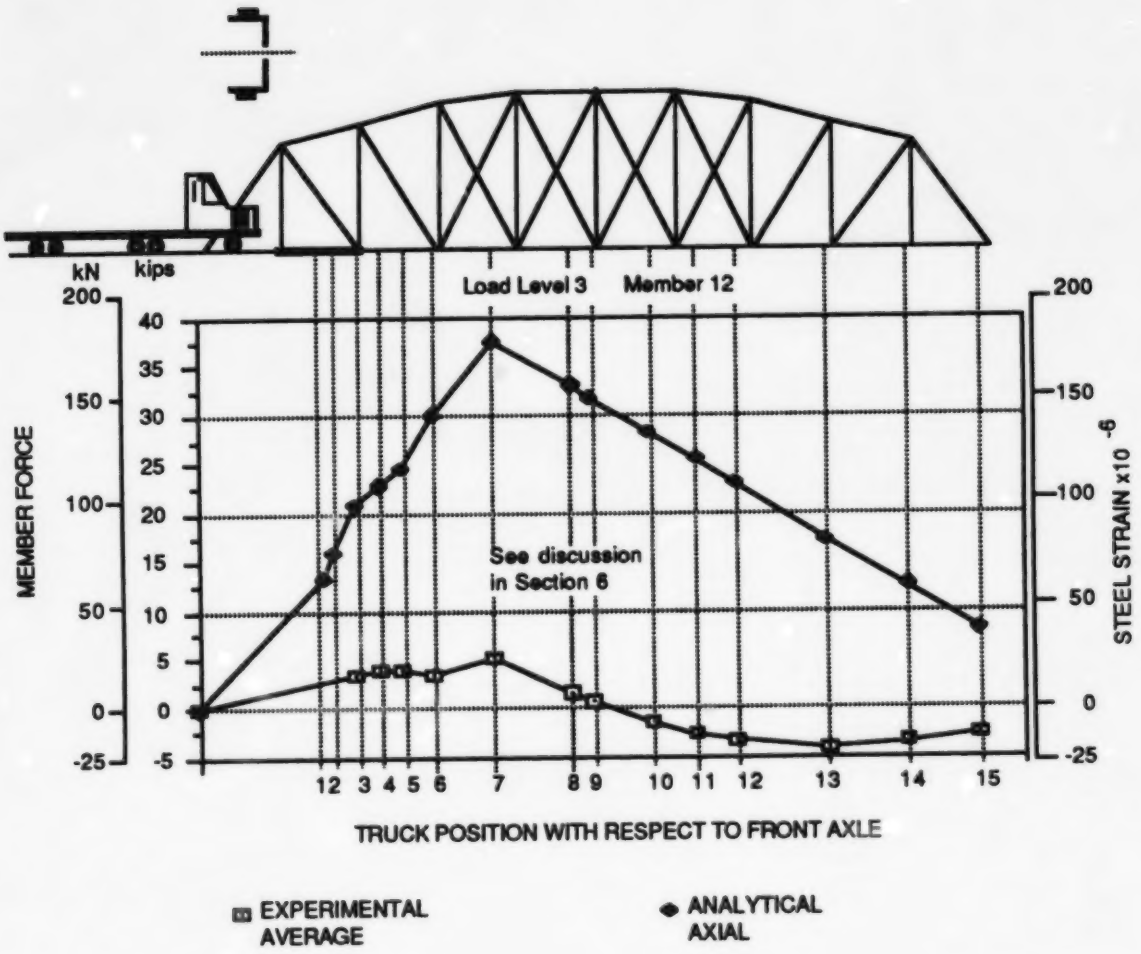


Figure 23/ Force Effects in Member 12

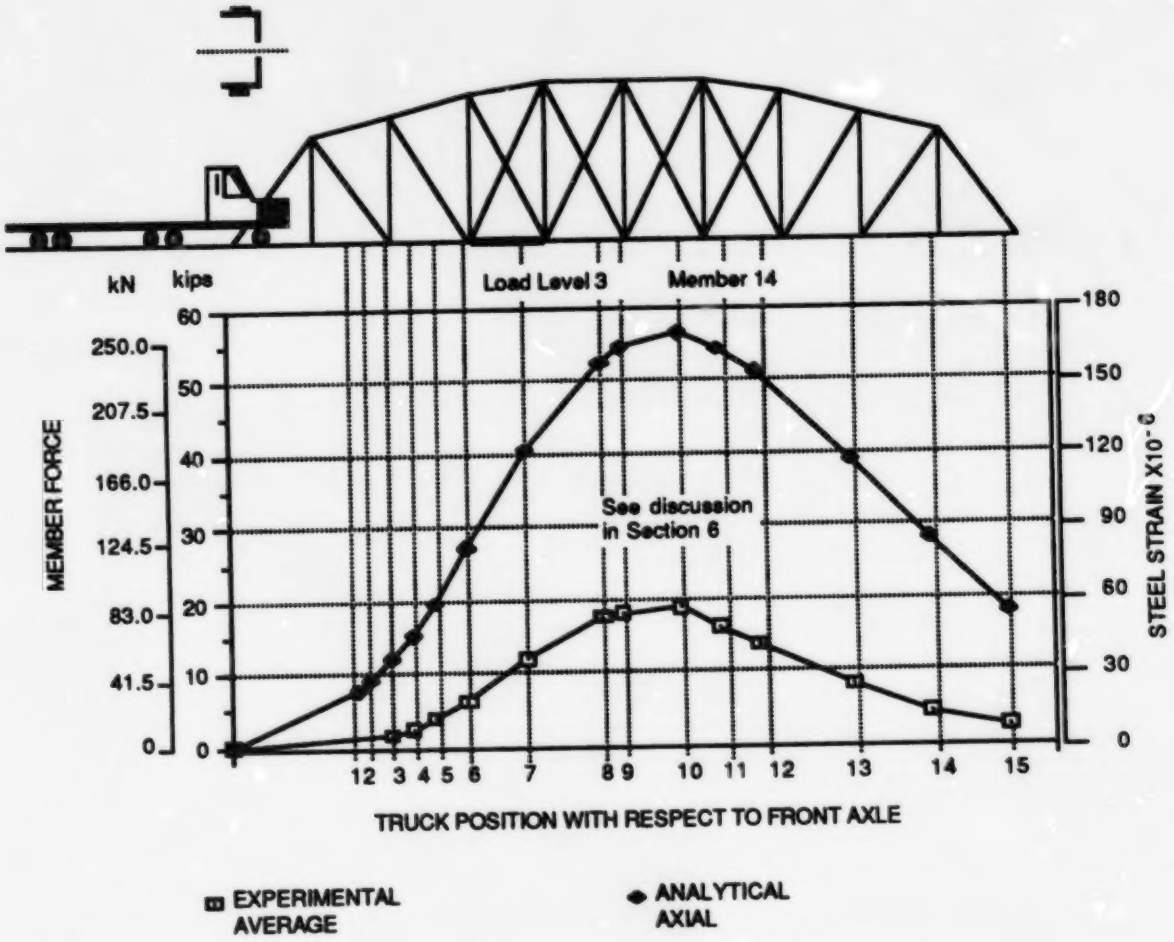


Figure 24/ Force Effects in Member 14

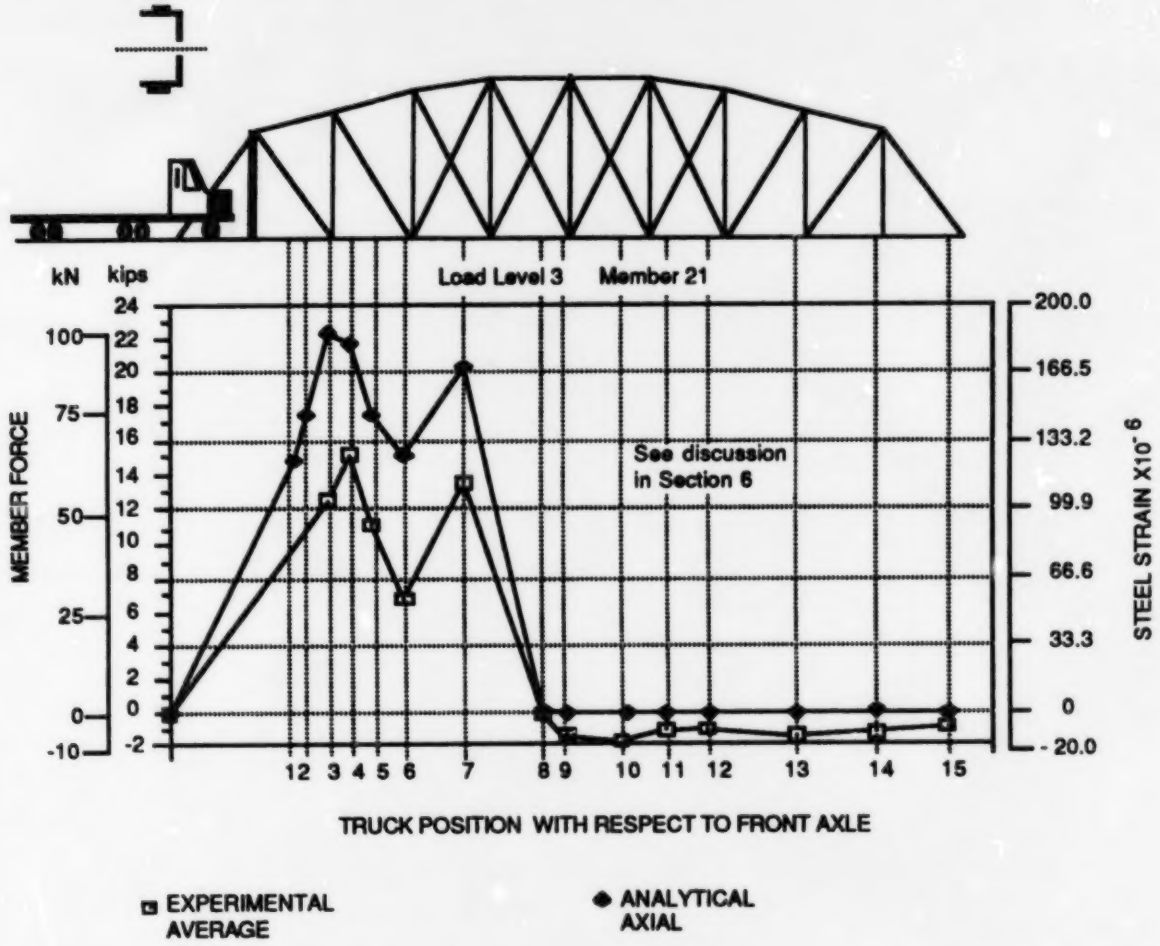


Figure 25/ Force Effects in Member 21

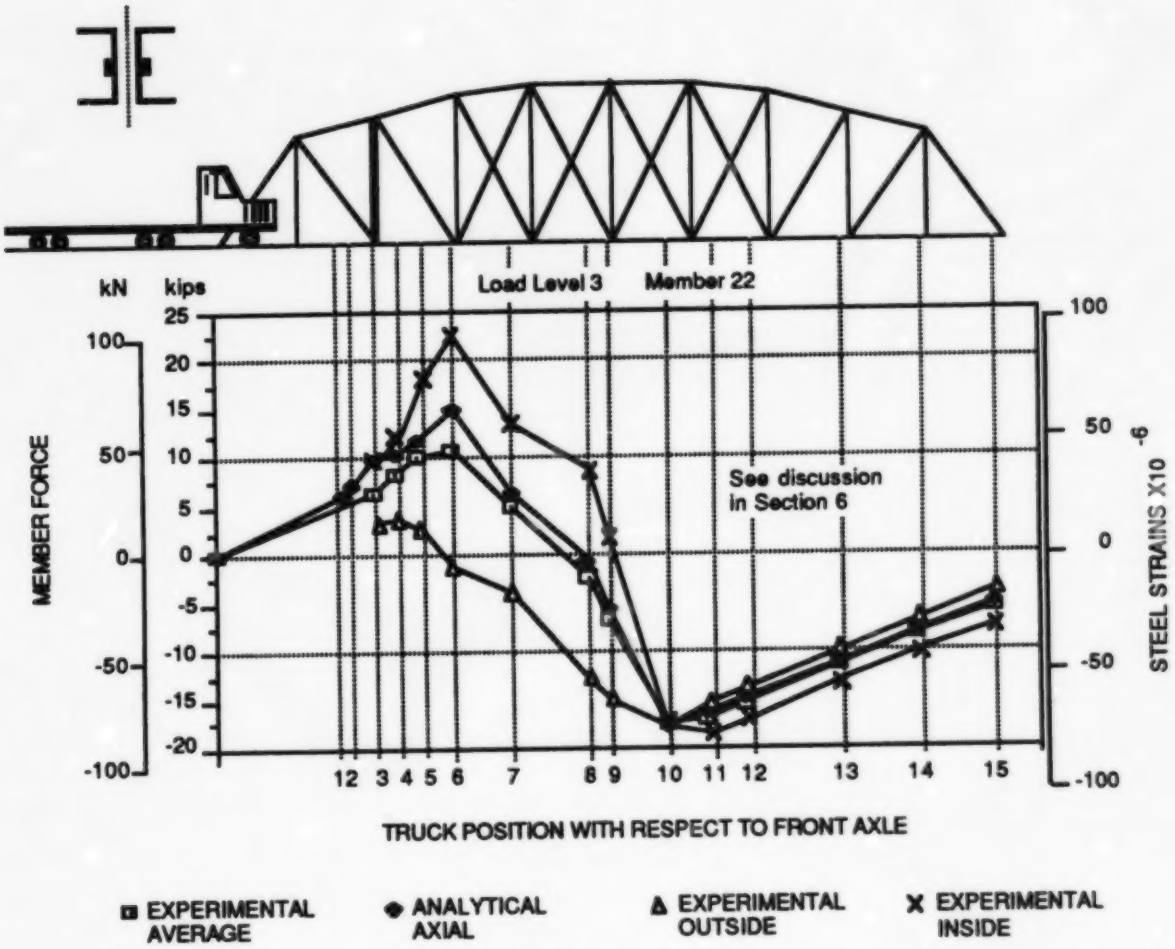


Figure 26/ Force Effects in Member 22

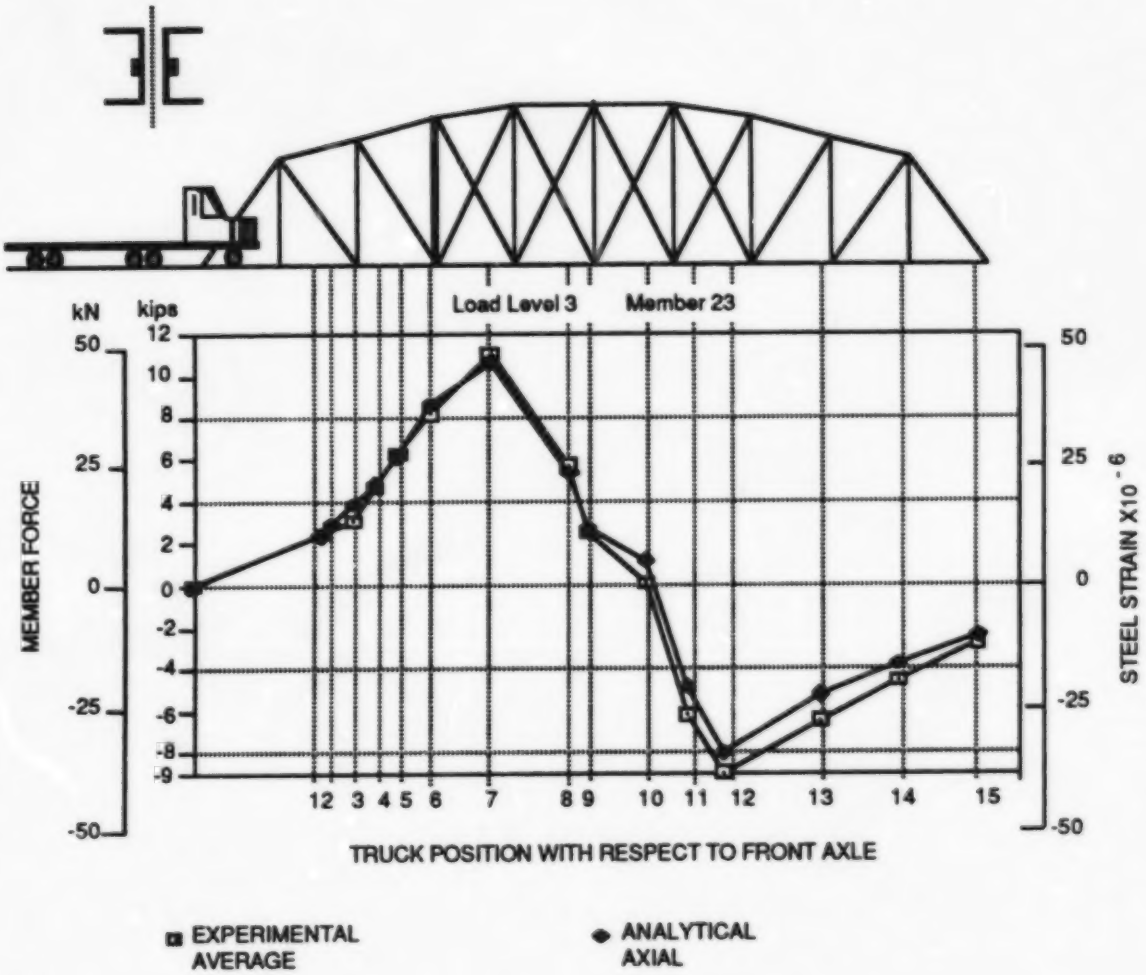


Figure 27/ Force Effects in Member 23

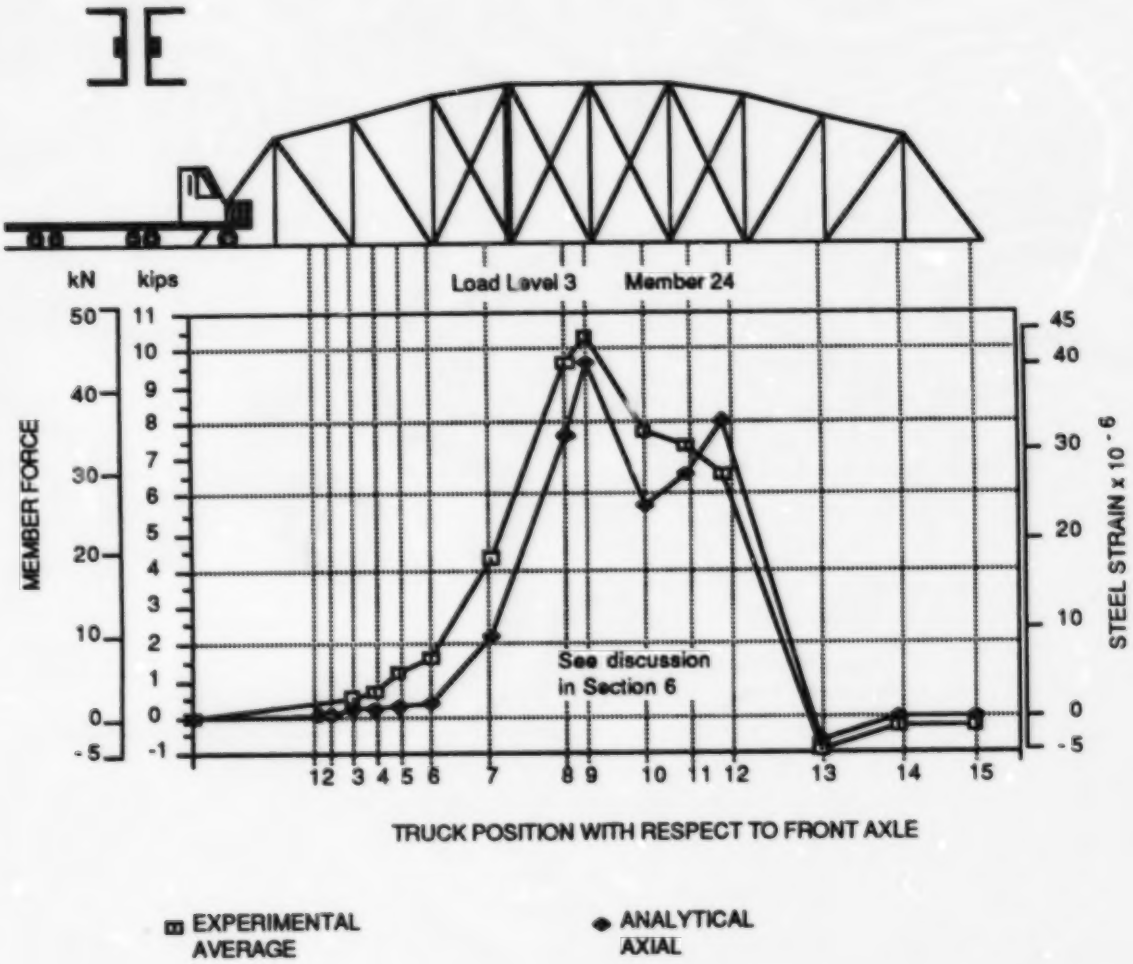


Figure 28/ Force Effects in Member 24

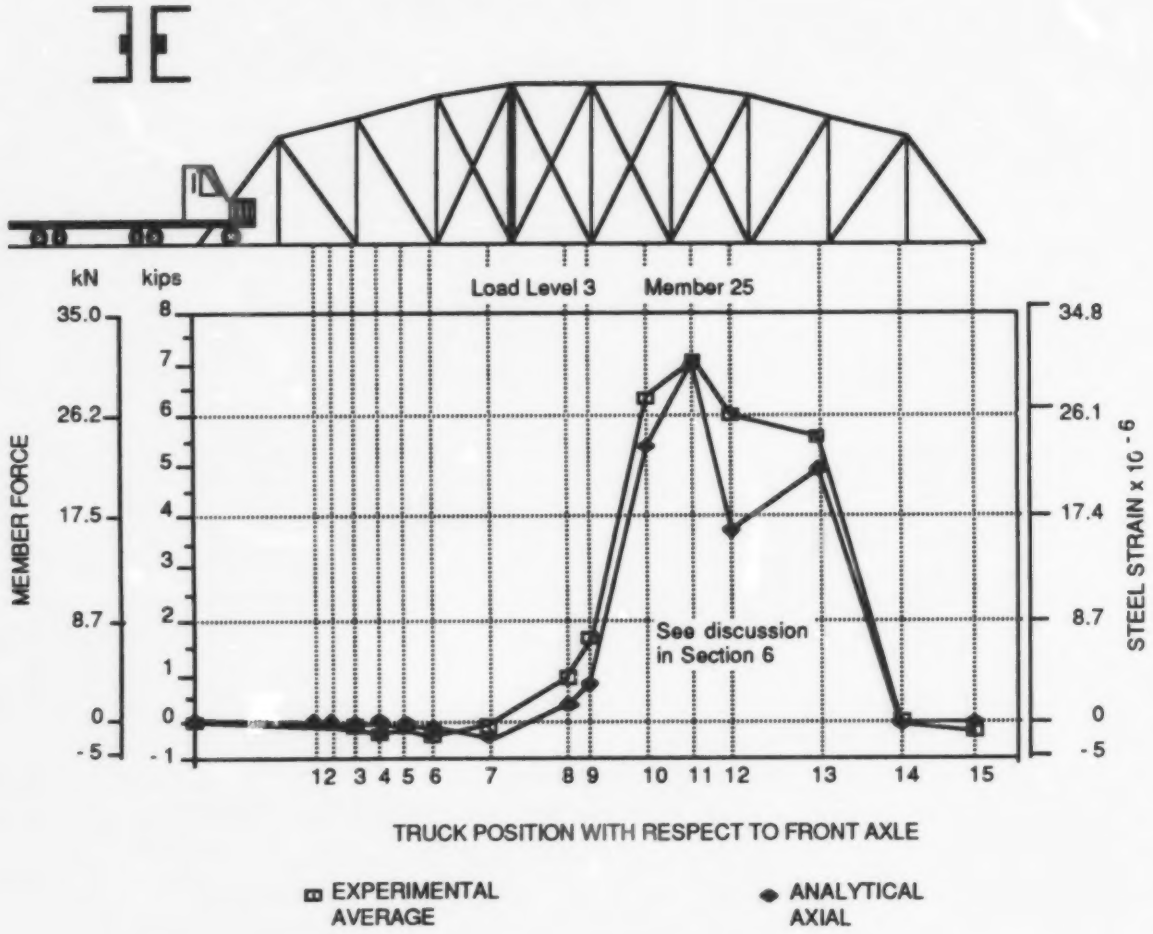


Figure 29/ Force Effects in Member 25

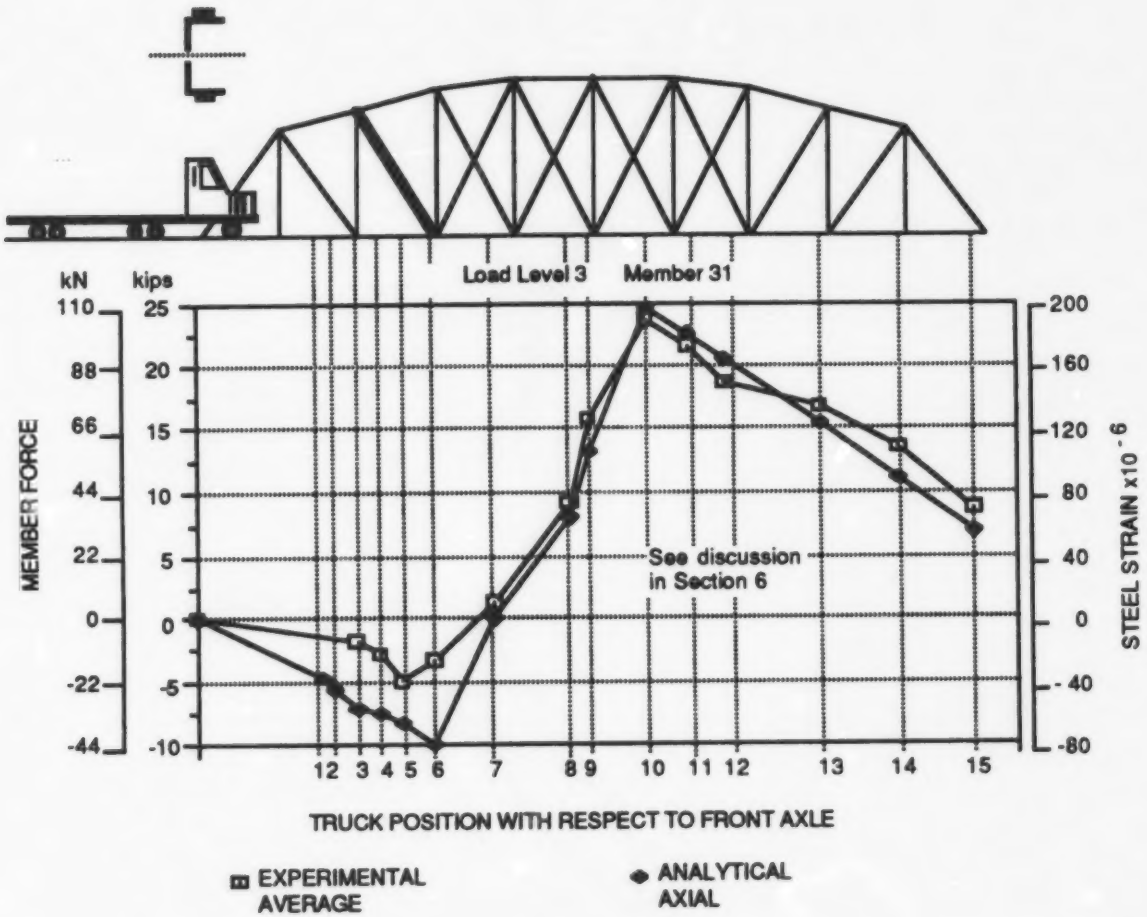


Figure 30/ Force Effects in Member 31

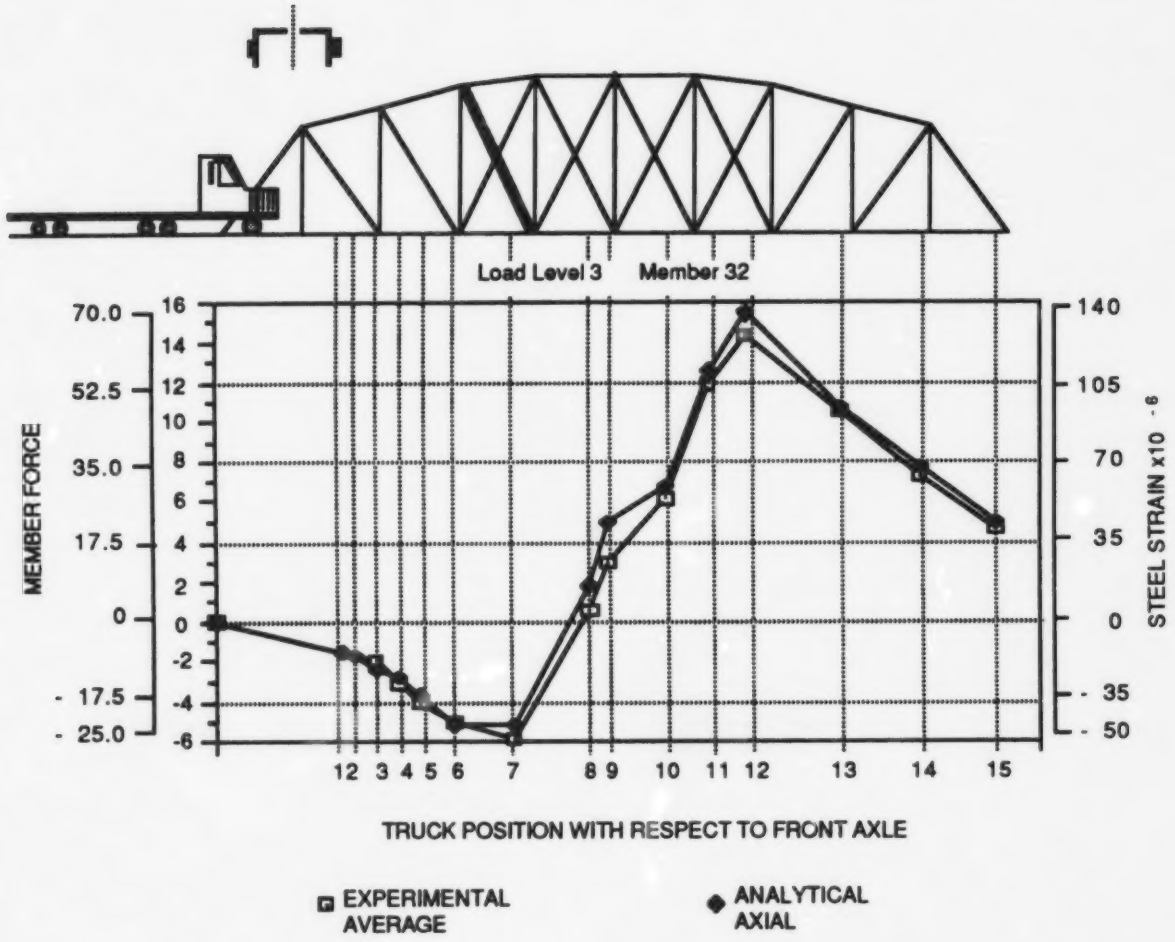


Figure 31/ Force Effects in Member 32

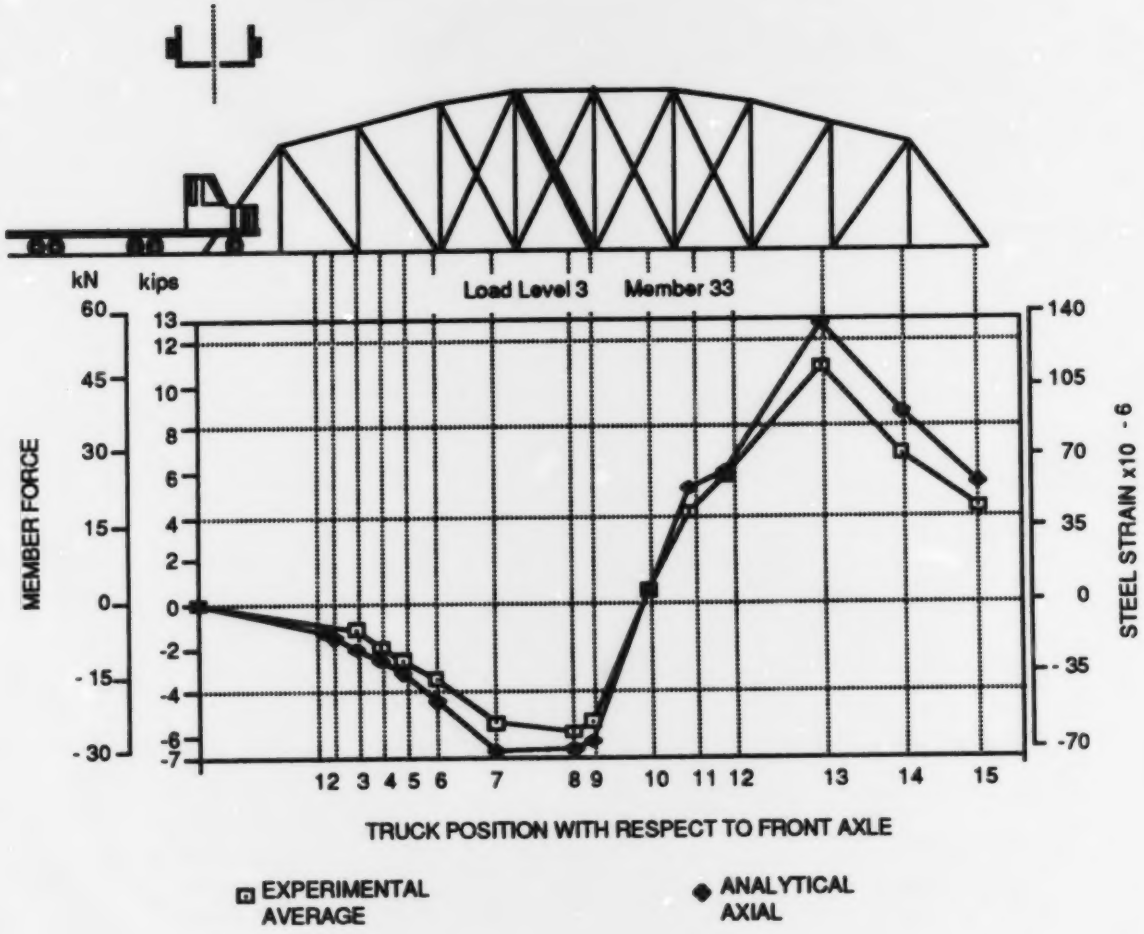


Figure 32/ Force Effects in Member 33

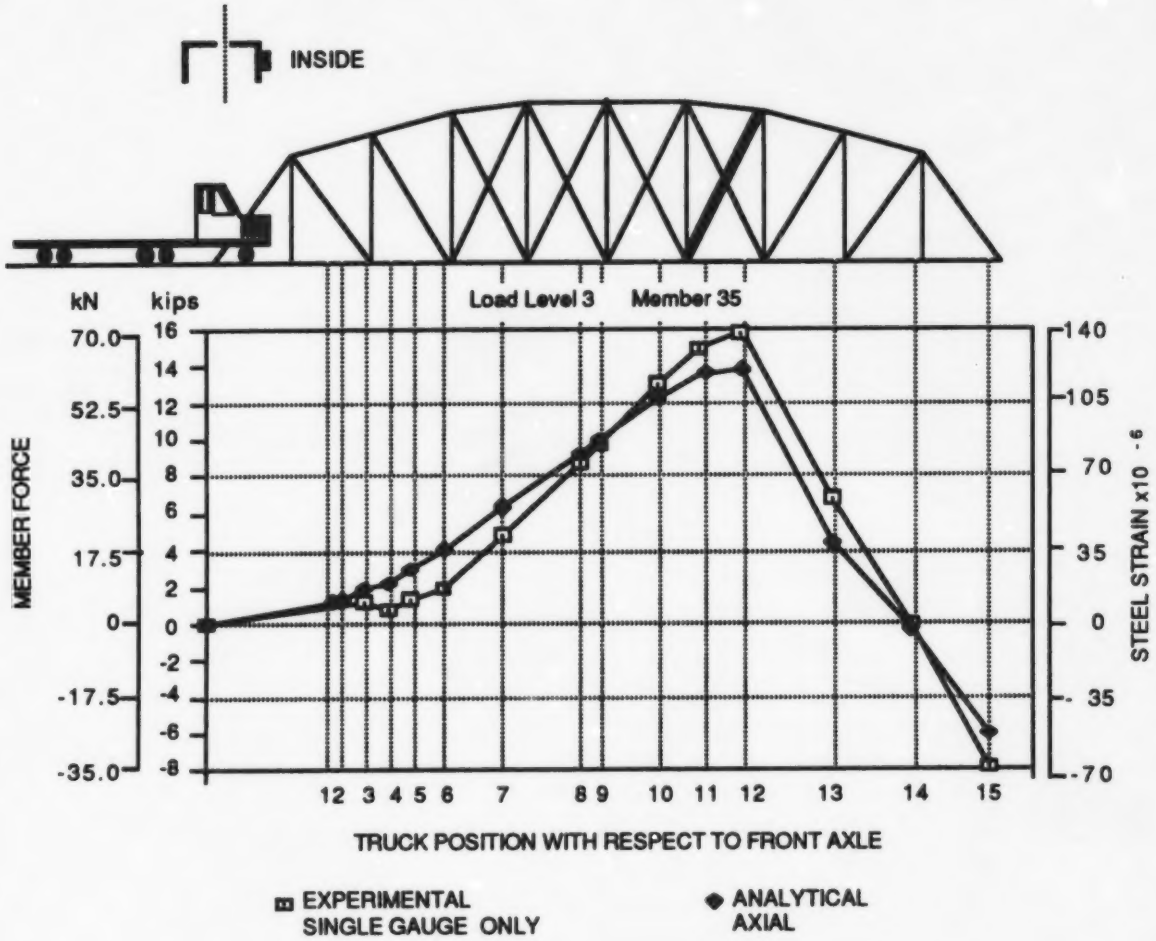


Figure 33/ Force Effects in Member 35

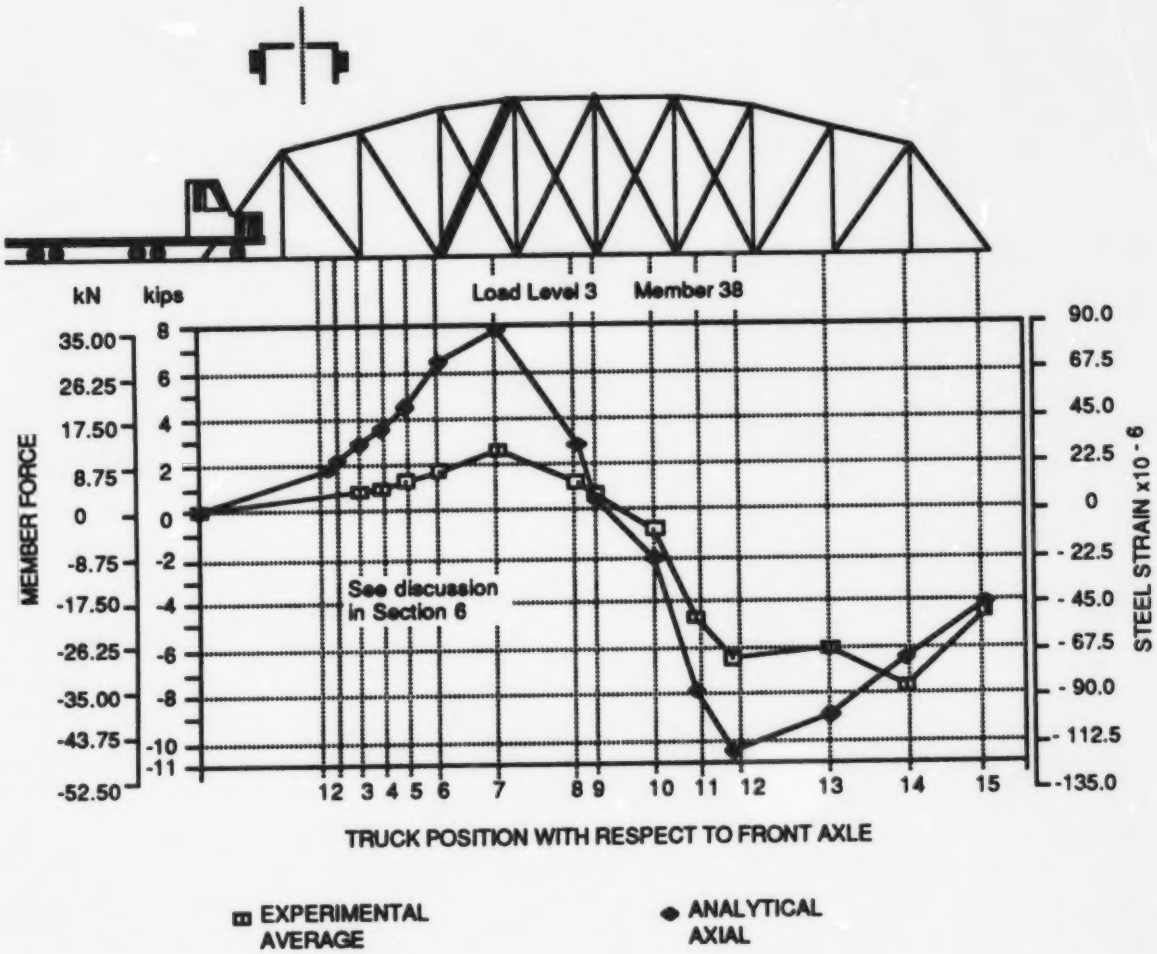


Figure 34/ Force Effects in Member 38

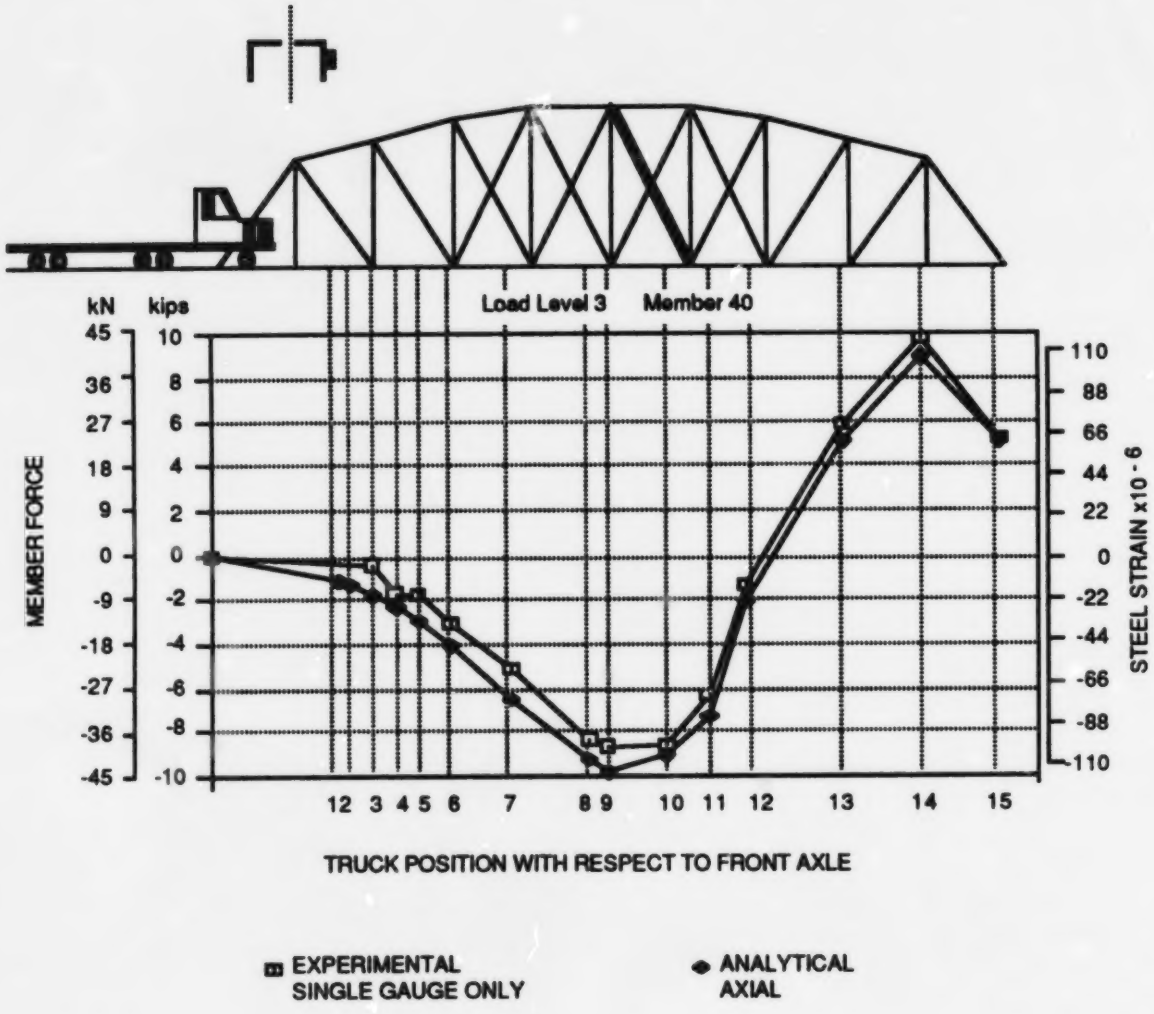


Figure 35/ Force Effects in Member 40

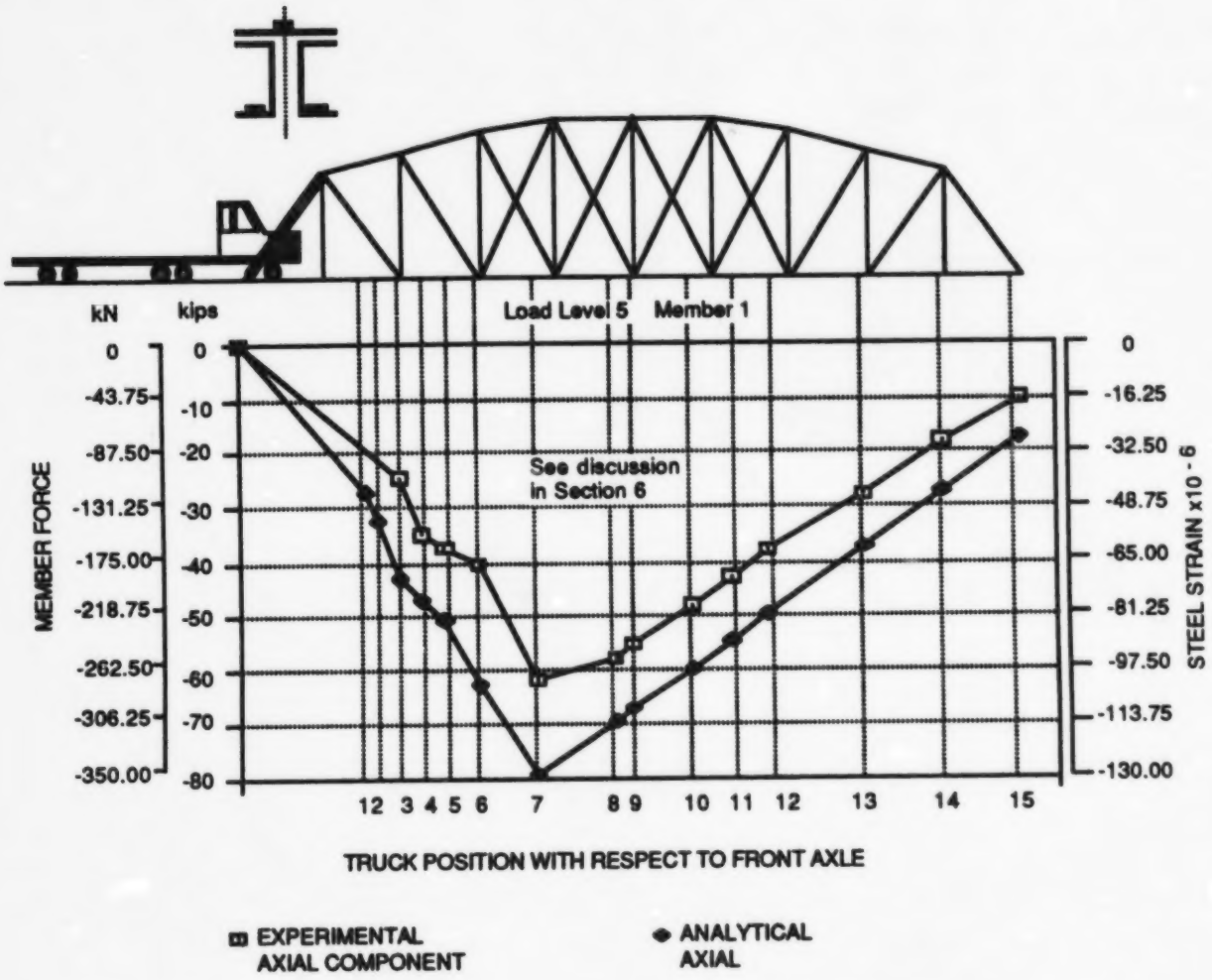


Figure 36/ Force Effects in Member 1

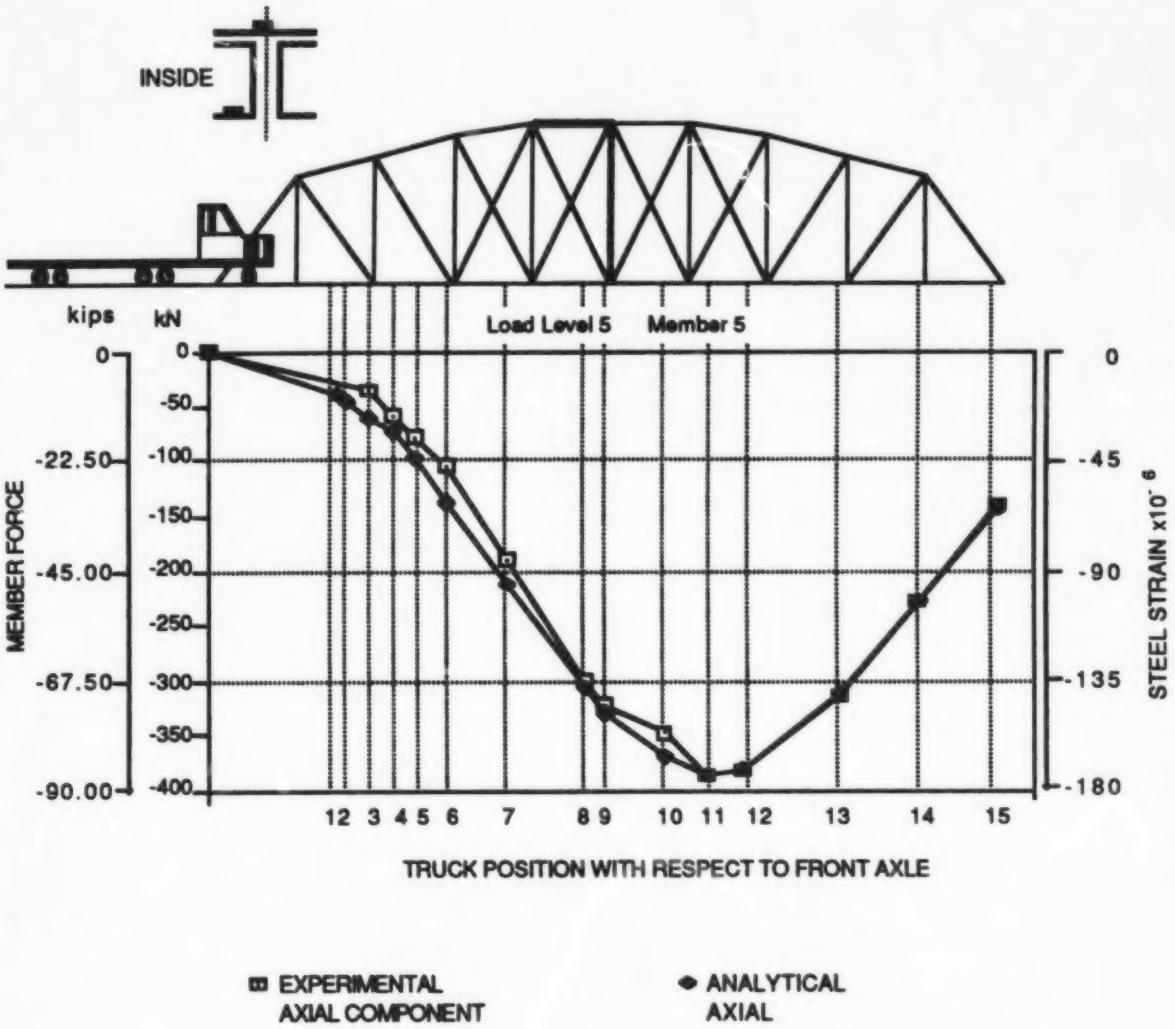


Figure 37/ Force Effects in Member 5

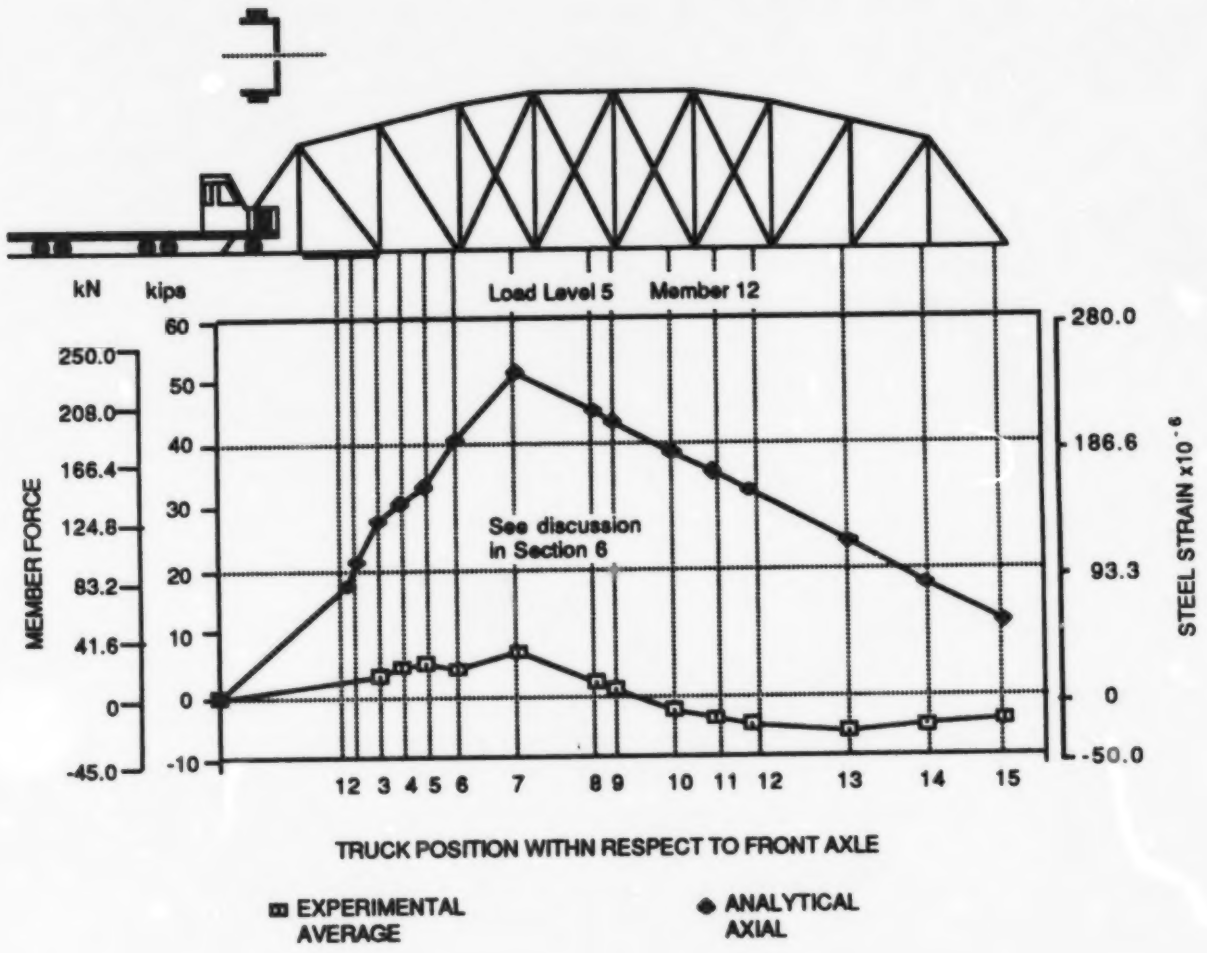


Figure 38/ Force Effects in Member 12

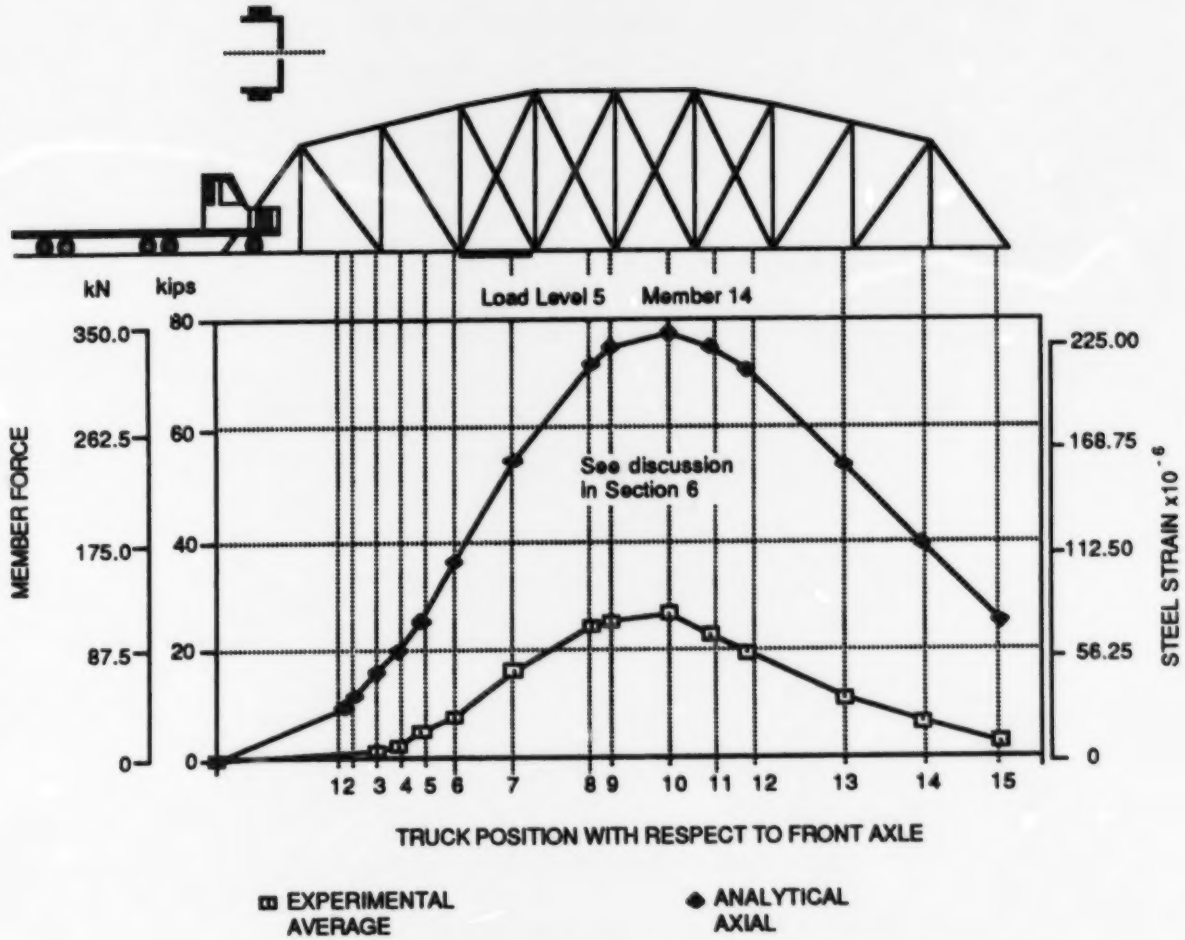


Figure 39/ Force Effects in Member 14

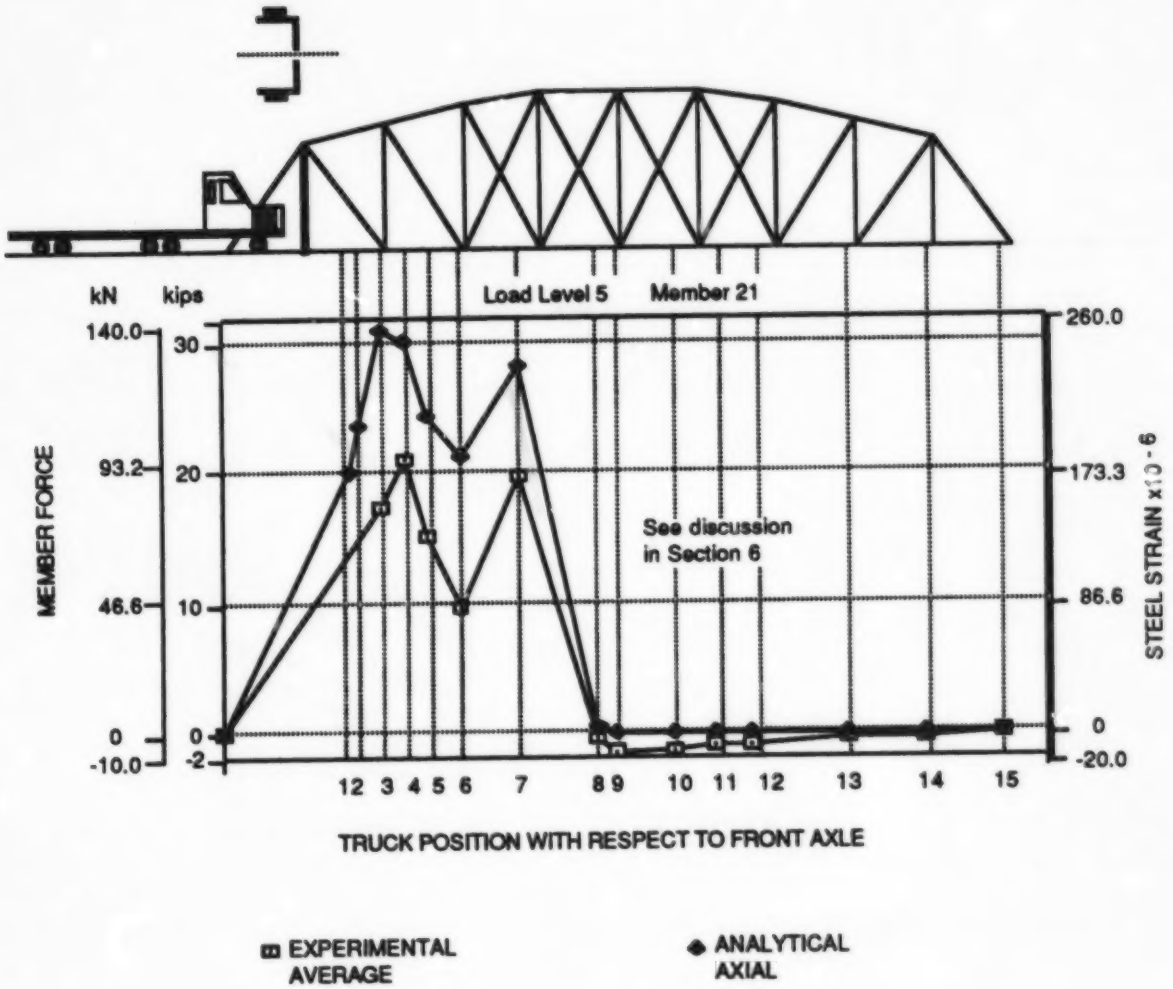


Figure 40/ Force Effects in Member 21

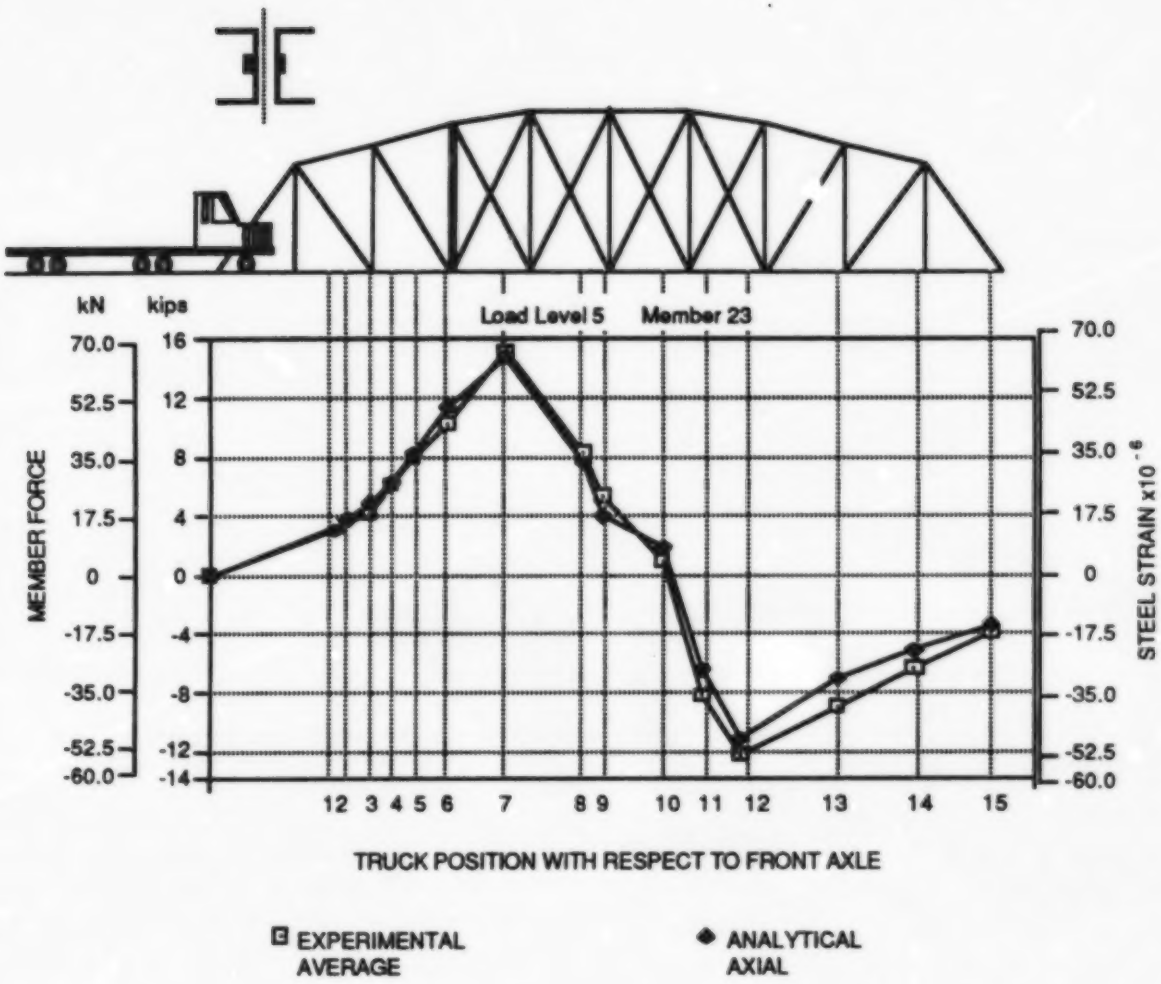


Figure 41/ Force Effects in Member 23

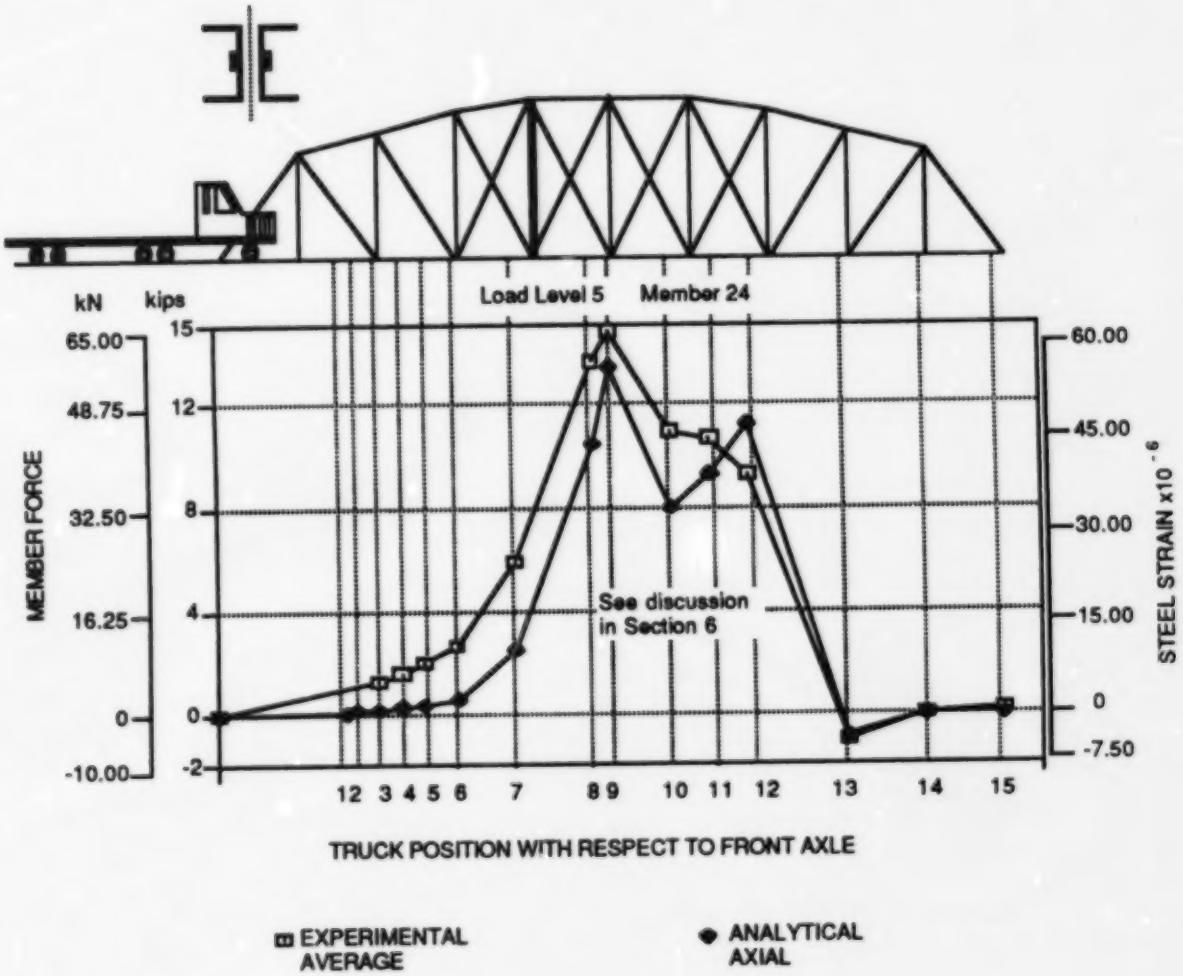


Figure 42/ Force Effects in Member 24

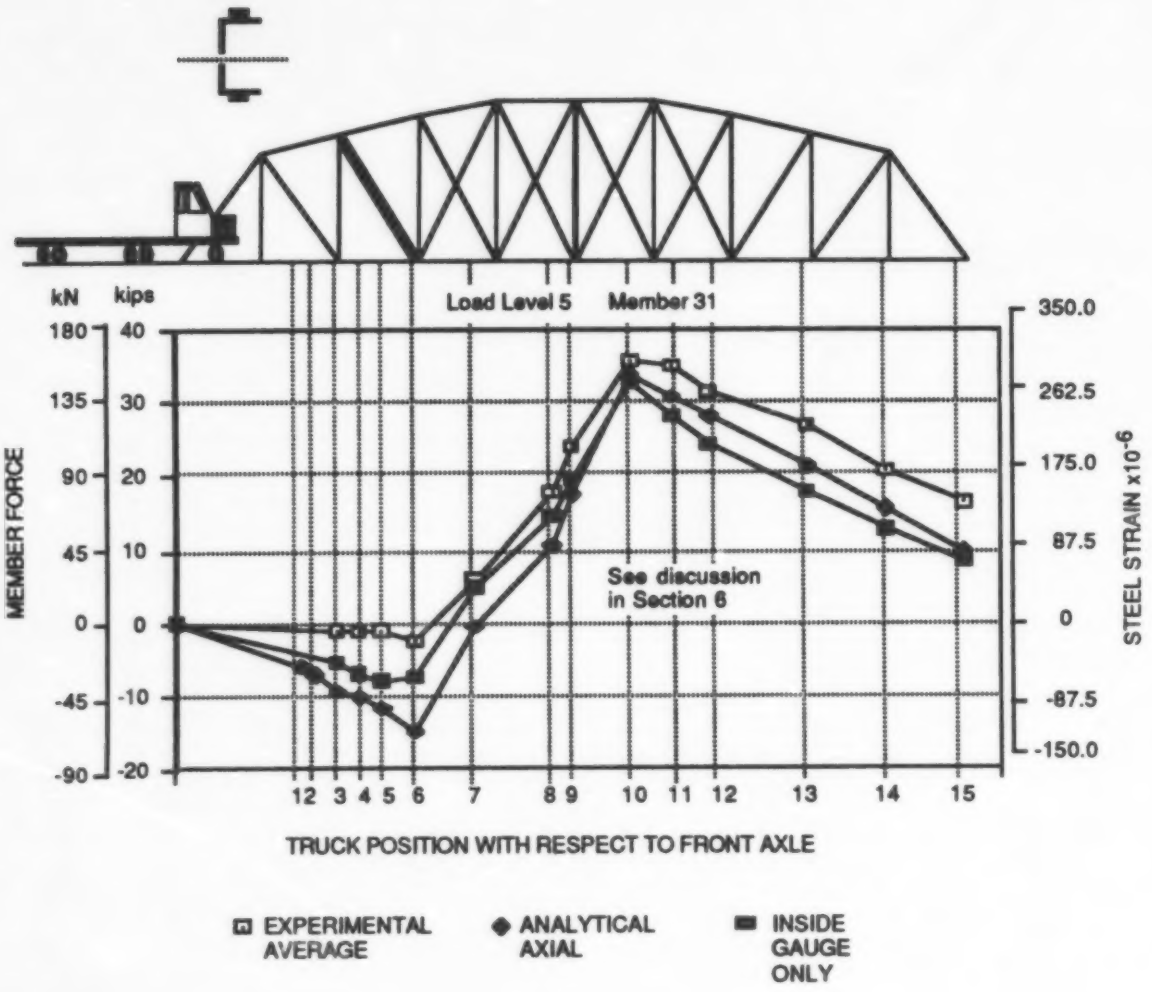


Figure 43/ Force Effects in Member 31

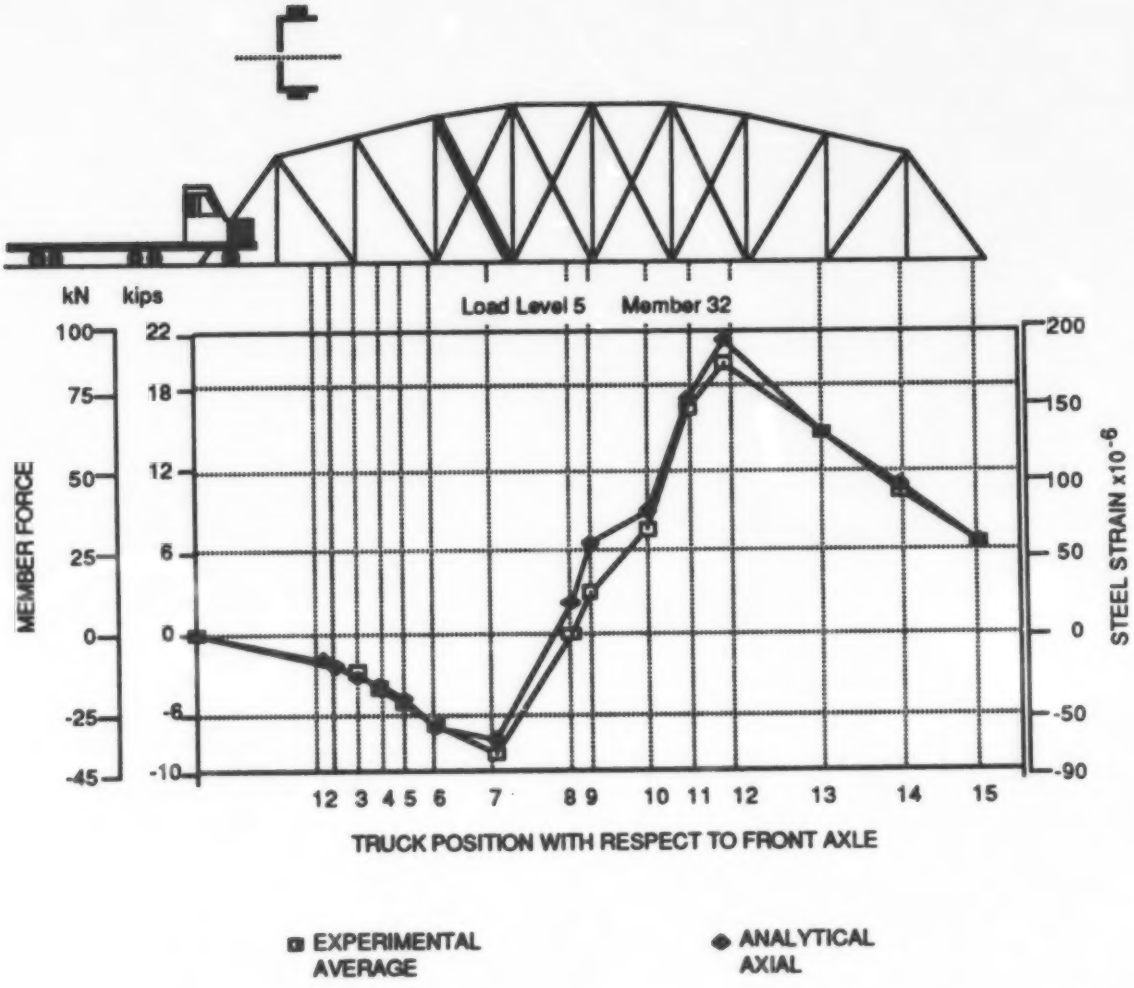


Figure 44/ Force Effects in Member 32

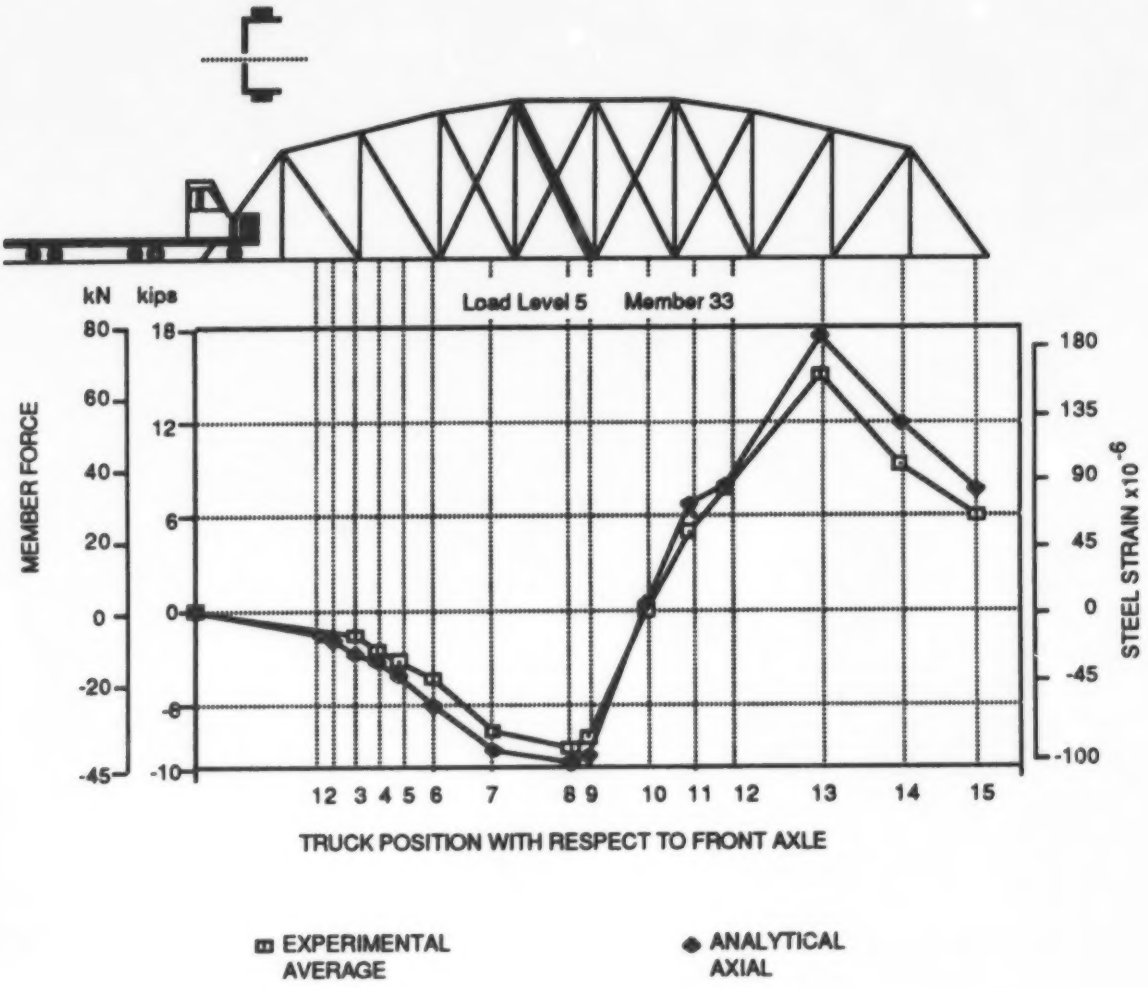

kN kips
Load Level 5 Member 33
MEMBER FORCE
STEEL STRAIN x10^-6
80 60 40 20 0 -20 -45
18 12 6 0 -6 -10
180 135 90 45 0 -45 -100
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15
TRUCK POSITION WITH RESPECT TO FRONT AXLE
EXPERIMENTAL AVERAGE
ANALYTICAL AXIAL

Figure 45/ Force Effects in Member 33

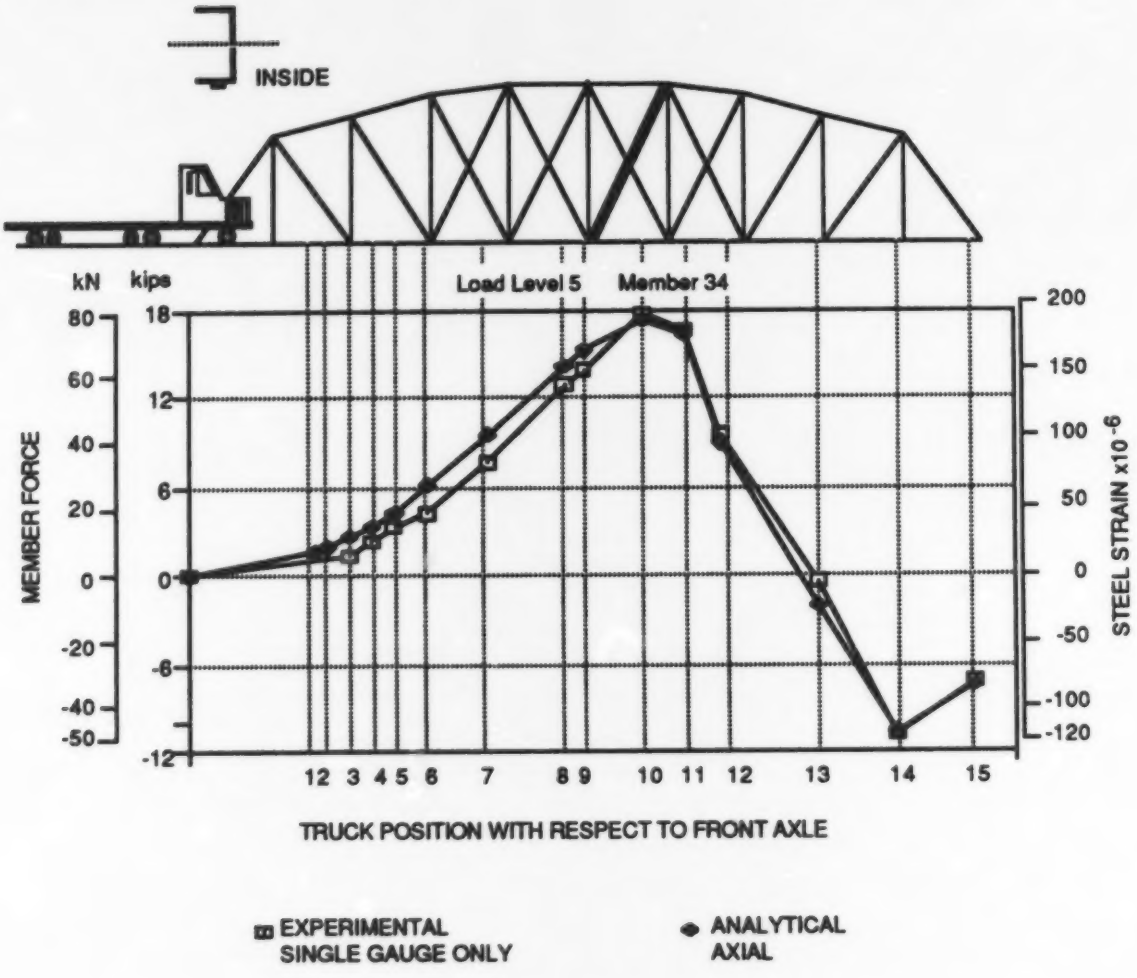


Figure 46/ Force Effects in Member 34

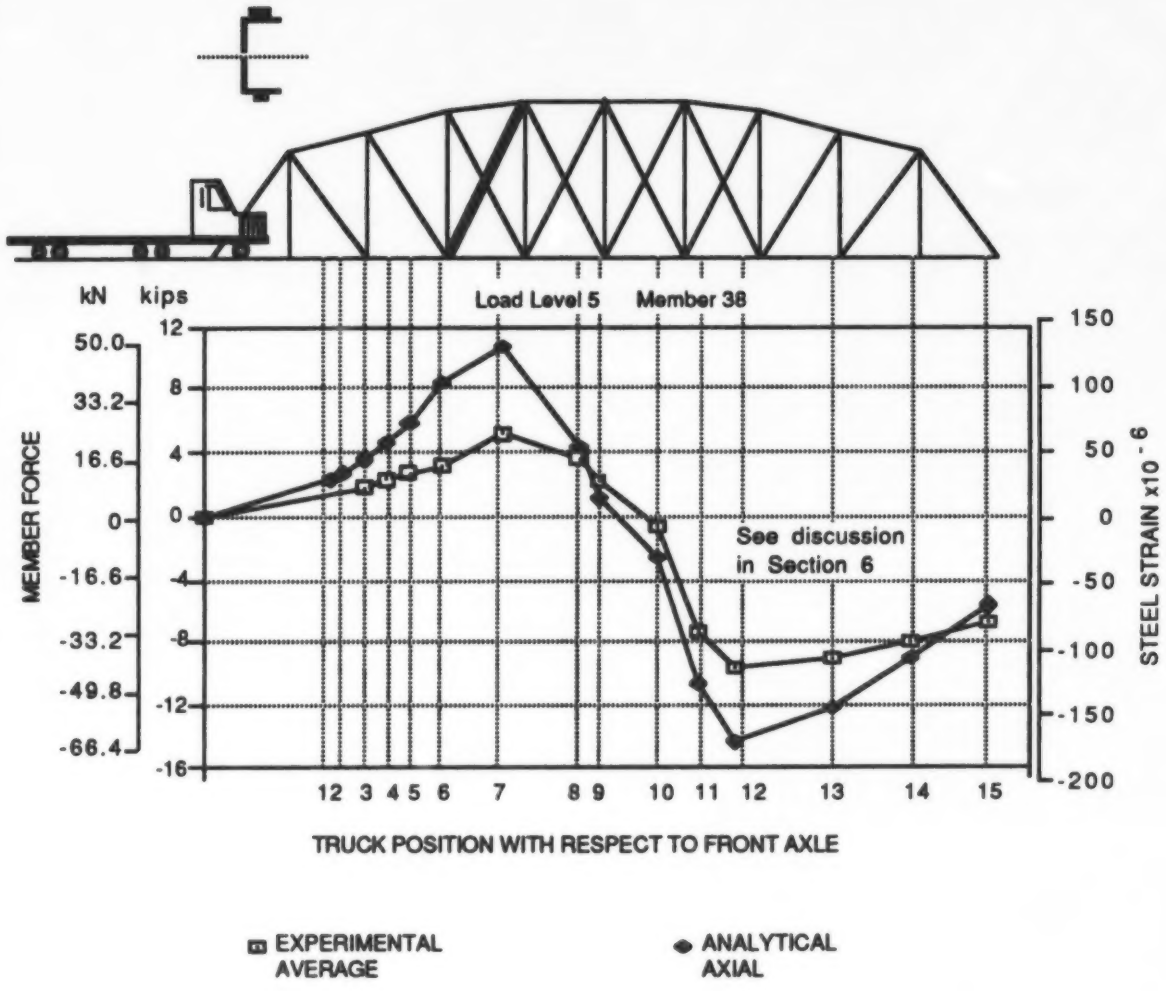


Figure 47/ Force Effects in Member 38